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CALCULATION PACKAGE COVER SHEET

Client: Gowanus Canal Remedial Design Group (RD Group) **Project:** Gowanus Canal Superfund Site **Project #:** HPH106A

TITLE OF PACKAGE: **ARMOR LAYER DESIGN**

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ARMOR LAYER DESIGN

INTRODUCTION AND PURPOSE

As outlined in the Record of Decision (ROD) (EPA, 2013), a multilayered capping system ("cap") will be constructed within the Canal to: (i) provide a layer at the bottom of the Canal that is physically stable and meets remedy performance criteria for contaminant of concerns (COCs); and (ii) prevent unacceptable amounts of contaminants, including dissolved-phase constituent and residual non-aqueous phase liquids (NAPLs) from migrating at a level that can pose risk to ecological receptors from beneath the cap to surface layers and Canal surface water.

The calculation package presented herein addresses the design of the armor layer in remediation target area (RTA) 1 and 4th St. Turning Basin (TB4) Pilot Study Area. The armor layer will be placed on top of the isolation and filter layer and is intended to (i) become the new sediment-water interface and (ii) provide physical stability to prevent erosion and/or material loss. The armor layer will consist of appropriately sized material to withstand erosional forces of the flushing tunnel and vessel traffic according to ROD requirements (EPA, 2013). Gravel will be placed within the voids of the armor layer. The isolation and filter layer underlying the armor layer will consist of sand. The gravel material placed within the voids of the armor layer and sand in the underlying isolation and filter layer will provide an ecological habitat layer to "facilitate benthic recolonization" (EPA, 2013). The design of the ecological habitat layer and the treatment layer are addressed as separate calculation packages and are provided as Appendix B11 and B9, respectively.. In addition to evaluating erosional forces on the armoring layer, this calculation package also evaluates if the potential for ice scour is a design concern. A schematic of the overall cap design in TB4 Pilot Study area is provided in **Figure 1**. The cap design in RTA1 is anticipated to be similar, and will be updated based on lessons learned in the TB4 Pilot Study

The design presented herein consists of an evaluation of traditional engineering methods for armoring (i.e., riprap) and alternative lining materials such as marine mattresses, articulated concrete blocks (ACBs), and fabric-formed concrete. The selection of a material would be dependent on the constructability, ability to limit erosion due to navigation impacts and hydrodynamic forces, potential environmental impacts, cost efficiency, and durability during periodic maintenance dredging relative to riprap.

A figure presenting the extent of RTA1 and TB4 Pilot Study Area is provided as **Figure 2**.

DESIGN CRITERIA AND KEY INPUTS

The selection of an appropriate riprap gradation for armoring is based on three primary components which includes: (i) erosional forces due to vessel traffic; (ii) erosional forces due to hydrodynamic forces (i.e., flushing tunnel, storm surge); and (iii) potential erosion due to ice scour. To adequately design for these components, the selected design criteria are as follows:

Design Life – The cap design life was selected to be 100 years.

Bottom of Cap Elevation – The bottom of cap elevation was assumed to be at the same elevation as the bottom of soft sediment in RTA1.

In TB4 Pilot Study Area, the bottom of cap elevation is between -15-ft and -16-ft NAVD88.

Top of Armor Elevation **–** For the armoring design in RTA1, it was assumed the top of armor was approximately four feet (ft) above the bottom of soft sediment in RTA1. Thus, the top of armor elevation in RTA 1 is anticipated to range between -12-ft North American Vertical Datum of 1988 (NAVD88) and -20-ft NAVD88.

For the 100% TB4 Pilot Study Design, the top of cap elevation will be between -12.7-ft and 13.7-ft NAVD88 in theTB4 Pilot Study Area.

Overdredge Allowance – Up to six inches of overdredge allowance will be provided for the RTA1 and TB4 design, however, this was not included in the analysis presented herein as analyzing the top of armor at a higher elevation is considered conservative for the purposes of the armoring design.

Navigational Elevation – The navigational elevation for RTA1 is assumed to be -7.77-ft NAVD88 (Geosyntec, 2016a, b). The navigational elevation will be higher than the top of cap. Although not required, there will be space below the navigational elevation in RTA1 to allow for storage of future sediment accumulation, however, this does not directly affect the design of the armoring layer. A navigational elevation in TB4 has not been established, however, there will be sufficient underkeel clearance relative to the top of cap for the anticipated vessels.

Tidal Elevation - The ROD states the "final dredge depth would need to ensure that the final sediment surface remains submerged throughout the tidal cycle and minimize remedy implementation challenges (e.g., allow sufficient water depth for construction work throughout the tidal cycle)." Thus, to allow for cap maintenance, monitoring, and future construction, it was assumed vessels may work during low tides. Hence, the tidal elevation selected for the propeller wash evaluation was the mean lower low water [Mean Lower Low Water (MLLW) = -2.77-ft NAVD88] (NOAA, 2015).

Vessel Traffic – Information on existing vessel traffic in the Canal is primarily based on Baird's *Vessel Impacts Study Report* (2017). RTA1 has not been used for commercial navigation since 2008 (Baird, 2017) and TB4 is not currently used for commercial navigation. Future vessel traffic in RTA1 and TB4 is anticipated to be associated with Canal remediation, maintenance, and monitoring. Tugs with drafts ranging from four to six feet were assumed to be used for remediation, maintenance, and monitoring. These tugs were selected as they were presumed to be of adequate size to tow the anticipated barges and scows in RTA1 and TB4 and would have shallow enough draft to operate throughout the tidal cycle. Representative tugs used for the analysis include the: (i) 6 ft loaded draft, 700 horsepower (hp) "Rochelle Kaye"; (ii) 4 ft loaded draft, 660 hp "Gabby Miller"; and (iii) 4.5 ft loaded draft, 500 hp "Clyde" tug. More specific details on each of the tugs analyzed are provided within **Table 1**.

Underkeel Clearance - The three tugs have underkeel clearances ranging from 3.33-ft to 5.33-ft at MLLW, where the top of armor elevation is at an assumed maximum in RTA1 of -12-ft NAVD88. In TB4, the underkeel clearance would be approximately 4-ft to 7-ft for the analyzed tugs (top of cap elevation = -12.7-ft to -13.7-ft NAVD88). Since the underkeel clearance is larger than the typical design recommendations (2 ft or 10% of vessel draft), itwas not evaluated further. In locations where the bottom of soft sediment is deeper or the cap is thinner, the underkeel clearance would be greater.

Riprap Gradation and Thickness – If riprap is selected, then the minimum riprap thickness will be the greater of two (2) times the median stone size (d_{50}) or 1.5 times the maximum stone size (d_{100}), based on subaqueous capping guidance by the United States Army Corps. of Engineers (USACE) for EPA (EPA, 1998) for propeller wash armoring design.

Factor of Safety (FS) – The FS was selected as 1.5 and utilized for "bed shear stress" calculations.

METHODOLOGY AND DESIGN PARAMETERS

The methodology and design parameters for analyzing propeller wash, hydrodynamic forces, and ice scour are described in the following subsections:

Propeller Wash

Previous Studies

Multiple reports have previously analyzed propeller wash within the Canal (Baird, 2017; CH2M, 2011, and CH2M, 2015). Generally, these reports focused on the potential for impacts to the Canal bed due to propeller wash in RTA2 and 3, since it was assumed vessel traffic would be limited in RTA1. Commercial traffic is also not currently present in TB4, however, much of the methodology and analysis can be applied to analyze vessel impacts in both RTA1 and TB4. A brief summary of the three design reports is provided below:

- **Baird (2017)** – A major component of the report focuses on estimating propeller wash induced near-bed velocities for tugs that frequent the lower and middle reaches of the Canal. The tugs analyzed are used for commercial traffic and would be larger than what is anticipated in RTA1 and TB4 for maintenance, monitoring, and future construction. The methodology utilized to estimate the velocities were from EAU (1996) and PIANC (1997). For the analyzed tugs, they found that "armor material the size of boulders would be required to protect against wash-induced damage at lower tide levels."

- **CH2M (2015) –** In 2015, CH2M issued a technical memorandum for EPA which evaluated the applicability of using riprap and alternative armoring materials (e.g., marine mattresses, ACBs) in the cap design. The near-bed velocities utilized in the analysis were based on the 2012 vessel impacts study completed by Baird (2012b). The CH2M report recommended utilizing alternative lining material in RTA2 due to the relatively large stone sizes [i.e., median stone size (d_{50}) of 3 feet] that would be required to protect the cap from propeller wash. CH2M stated that the benefits of using an alternative lining material in RTA2 would include having a thinner cap allowing for limited dredging of native sediments (i.e., glacial deposits and native alluvial sediments) and deeper navigational depths. Propeller wash specifically in RTA1 and TB4 Pilot Study Area was not analyzed since commercial traffic is not planned and thus "propeller wash in these sections is not expected to be significant."

- **CH2M (2011)** – The analysis "Propeller Wash and Cap Armor Thickness Calculations" was completed by CH2M as part of the Feasibility Study in 2011 (CH2M, 2011). The near-bed velocities and bottom shear were based on the methods presented in Verhey (1983) and Blaauw and van de Kaa (1978). The riprap size was calculated for select commercial vessels frequenting RTA2 and 3 and methods presented in EPA guidance document on in-situ subaqueous capping (EPA, 1998). A propeller wash analysis specific to RTA1 and TB4 Pilot Study Area was not undertaken.

Although the three reports primarily focus on commercial vessel propeller wash impacts in RTA2 and 3, multiple design components are directly applicable to the design for areas without anticipated commercial vessel impacts, such as RTA1 and TB4 and are cited within the calculation package, where appropriate.

Near-bed velocity and Bed Shear Stress

Selection of an appropriate armoring design to mitigate the effects of propeller wash is typically based on near-bed velocities and/or bed shear stress. To estimate riprap sizes, both methods were utilized and to evaluate alternative lining, the selection of the appropriate method was dependent on available critical velocity and/or critical shear stress (i.e., velocity or shear stress to cause incipient motion) data from the manufacturer.

To estimate near-bed velocities in RTA1 and TB4 due to propeller wash, three different equations were initially evaluated including Blaauw and van de Kaa (1978), EAU (1996), and PIANC (1997). Of the three equations, only the EAU method is applicable to single and dual propeller vessels. Based on an initial analysis of single propeller vessels, it was found that the EAU method predicted higher near-bed velocities then the Blaauw and van de Kaa and PIANC methods. Thus, propeller wash from the selected vessels was evaluated using the EAU method since the analyzed tugs have dual propellers and the Blaauw and Van De Kaa and PIANC methods are only applicable to single propeller vessels.

As noted previously, a summary of the vessel specific input parameters is provided in **Table 1**. The process to estimate propeller jet velocities V_o (m/s), which are in turn used to estimate maximum near-bed velocities ($\max V_{bottom}$) is provided below:

The propeller jet velocity (V_o) is first calculated as:

$$V_o\,(m/s) = C_p \left(\frac{P}{\rho_w D^2}\right)^{1/3} \tag{1}$$

Where:

P is the screw (propeller) output or power (kW). EAU recommends selecting the speed to be 75% of the rated vessels speed for designing bottom protection measures. This corresponds to 42% of the vessel's rated power.

C_p is a coefficient equal to 1.48 for free screws without nozzles (i.e., non-ducted propellers) and 1.17 for screws in nozzle (i.e., ducted propellers);

ρ_w is the density of water, where the Canal was presumed to have a similar density as salt water (= 1.025 metric tons/m^3 , equivalent to a specific weight of 64 lbs/ft^3); and

D is the screw diameter (m).

Once the propeller jet velocity is calculated and converted to English units, the maximum near-bed velocity feet per second (fps) may then be calculated as follows:

$$\max V_{bottom} = V_o\, E \left(\frac{h_p}{D}\right)^a \qquad (2)$$

Where:

h_p = height of screw shaft above bottom = $z + (h - t)$

E = 0.71 for single screw-vessels with central rudder;

= 0.42 for single screw vessels without central rudder;

= 0.42 for twin-screw vessels with central rudder, valid for 0.9 < h_p/D < 3.0; and

= 0.52 for twin-screw vessels with twin rudders located after the screws, valid for 0.9 < h_p/D < 3.0

a = -1.00 for single-screw vessels and -0.28 for twin-screw vessels

z = distance from the centerline of propeller to bottom of vessel, which was assumed to be half the propeller diameter for the evaluation (=$D/2$).

h = water depth (ft)

t = vessel draft ft)

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To estimate bed shear stresses from vessels, a preliminary/cursory evaluation similar to CH2M (2015) was completed, where the bed shear stress (τ_b) pounds per square feet (psf) on the bed was calculated as follows (Hayes, et al., 2010)

$$\tau_b = \frac{1}{2} C_f \rho_w {V_o}^2 \qquad (3)$$

Where:

C_f = bottom friction factor = $0.01(D/h_p)$ (Hayes, et al. 2010)

Riprap Sizing

Riprap sizes were estimated based on maximum near-bed velocities and bed shear stress methodology.

The required riprap size (d_{req}), presumed to be similar to the median riprap size (d_{50}) was estimated as follows (EAU, 1996):

$$d_{req}\ (\text{ft}) \geq \frac{max\, {V_{bottom}}^2}{B^2 g} \frac{\rho_w}{(\rho_o - \rho_w)} \qquad (4)$$

Where:

B is a stability coefficient equal to: 0.90 for stern screws without central rudders; 1.25 for stern screws with central rudders, and 1.20 for bow thrusters;

g = gravitational acceleration of (= 32.2 ft/s^2); and

ρ_o = density of stone (= 4.65 slugs/ft^3, equivalent to a specific weight of 150 lbs/ft^3)

The median riprap size based on bed shear stress, may be calculated using the following equation developed based on the Shields diagram (FHWA, 2012):

$$d_{50} = \frac{\tau_b}{K_s g(\rho_o - \rho_w)} \qquad (5)$$

Where:

K_s = Shields parameter. Where Blaauw, et al. (1984) recommends a value of 0.03 for "practically no transport of riprap" and FHWA (2012) recommends a value of 0.03 for gravel and cobble sizes (*FHWA did not recommend a Shields parameter specifically for larger material*)

Sensitivity Analysis of Riprap Sizes

To evaluate the effects of key input parameters on the estimated riprap sizes and aid in the selection of these parameters, a sensitivity analysis was performed by varying vessel power, selected FS, and the top of cap elevation. In total, seven scenarios were analyzed, which included five evaluations using the modified FHWA method and two based on the EAU method. For the modified FHWA method, the five following scenarios were analyzed at various cap elevations: (i) 42% vessel power, FS = 1.2; (ii) 42% vessel power, FS = 1.5; (iii) 100% vessel power, FS = 1.0; (iv) 100% vessel power, FS = 1.2; (v) 100% vessel power, FS = 1.5. For the EAU method, two scenarios were analyzed where the vessel operated at 42% or 100% power for various cap elevations. Since the EAU method estimates a required stone size instead of a median stone size, a FS was not applied to the equation.

Alternative Lining Evaluations

Evaluations for alternative linings were completed for: (i) ACBs manufactured by Contech, Synthetex, and Shoretec; and (ii) fabric-formed concrete linings based on manufacturer or industry testing guidance. Permissible shear stresses or velocities recommended by the manufacturers were then compared to the calculated shear stresses or near-bed velocities to evaluate if the material was considered appropriate. Permissible shear stresses or velocities from the manufacturer were based on hydraulic flume testing.

In addition to evaluating ACBs and fabric-formed concrete linings, marine mattresses were also evaluated based on the permissible shear stress (τ_p) equation recommended by Tensar International Corp. (2016), a manufacturer of marine mattresses. The permissible shear stress equation applicable for marine mattresses is as follows:

$$\tau_p(psf) = 0.0091g(\rho_o - \rho_w)(MT + C) \qquad (6)$$

Where:

MT = marine mattress thickness (ft), which typically range in thicknesses from 6 to 24 inches; and

C = thickness coefficient of 4.07 for English units.

Hydrodynamic Forces

Hydrodynamic characteristics (i.e., depth-averaged flow velocities, bed shear stresses) were analyzed by Baird (2012a) based on existing bathymetric conditions for multiple scenarios including: (i) existing conditions during neap tide with moored barges; (ii) existing conditions during spring tide with moored barges; (iii) existing conditions during spring tide (without barges); (iv) flushing tunnel operating during spring tide with moored barges; (v) flushing tunnel operating during spring tide without moored barges; and (vi) during Hurricane Irene. For the purposes of generally understanding if propeller wash or other hydrodynamic forces (i.e., flushing tunnel, storm surge) control the design of the armoring layer, this evaluation is considered adequate. If this analysis is updated prior to the 100% RTA1 design, it would need to account for: (i) variation in flushing tunnel flows based on tidal conditions (an average flushing tunnel flow of 215 million gallons per day (MGD) was utilized in Baird's analysis, which is the future target flow rate as stated in the Gowanus Canal Waterbody/Watershed Facility Plan Report [NYCDEP, 2008]); and (ii) utilize the top of cap surface instead of the bathymetric surface, which is anticipated to reduce shear stresses and velocities. Discharges from combined sewer overflows and stormwater outfalls were not accounted for in Baird (2012a) study, however, Baird qualitatively anticipated the flows to be smaller than the amount of flow due to the flushing tunnel (see Figure 1.1., Baird, 2012a). Discharges from the flushing tunnel have a limited effect on the TB4 design due to the sheltered nature of the Basin.

Ice Scour

Damage to the cap due to ice scour is not anticipated to be a significant design concern due to the area's climate and the dead-end nature of the Canal, however, ice thicknesses were estimated using the Stefan formula. The simplified equation to calculate ice thickness for fresh water, which freezes more readily than salt water or brackish water (as found in the Canal), is as follows (USACE, 2002):

$$x = \alpha {\emptyset_d}^{1/2} \tag{7}$$

Where:

x = ice thickness (inches)

α = empirical coefficient based on local conditions such as snow cover, winds, and solar radiation

$\emptyset_d$ = air freezing index (°F - days)

The empirical coefficient, α, was selected based on recommended values (0.20 to 0.40) for a "sheltered small river" ice cover condition. A "sheltered small river" condition was selected since Canal is sheltered and has some flows due to the flushing tunnel, tidal effects, and other discharges.

The air freezing index for New York City for a 100-year (yr) recurrence interval is 440 °F - days based on National Climate Data Center (NCDC) data for Central Park located approximately seven miles away from the Site (NCDC, 2016).

CALCULATIONS AND RESULTS

Detailed hydraulic calculations related to propeller wash and hydrodynamic forces are described below. In addition, the calculations and results of the ice scour analysis are also provided.

Propeller Wash

Near-bed velocity and Bed Shear Stress

Jet propeller velocity calculations were computed for three vessels (Rochelle Kaye, Gabby Miller, and Clyde tugs). Jet propeller velocities were found to range from 30 to 33 feet per second (fps) at 75% rated speed (42% power). The jet propeller velocities were then used to estimate near-bed velocities at a range of assumed top of armor elevations (= -12-ft to -20-ft NAVD88) at MLLW (= -2.77-ft NAVD88). The near-bed velocities on the armoring layer at an elevation of -12-ft NAVD88 ranged from 12 to 14 fps, which correspond to bed shear stresses of 6 to 9 psf with a FS of 1.5. A figure comparing the bed shear stresses for the three tugs using the EAU and FHWA methods is provided as **Figure 3**. Summary tables of the propeller wash calculations for the Rochelle Kaye, Gabby Miller, and Clyde tugs are provided as **Tables 2, 3, and 4**, respectively. Hand (manual) calculations verifying the tables are provided as **Attachment 1**.

Riprap Sizing

The EAU method estimated similar riprap sizes as the FHWA method (with a FS of 1.5) at shallower depths (i.e., higher top of armor elevations). At greater depths, i.e. lower top of armor elevations, the EAU method predicted larger riprap sizes. At a depth of approximately 9 feet (top of armor elevation = -12-ft NAVD88), the EAU and FHWA method estimated required riprap sizes ranging from 2.2 to 3.5 ft for the three tugs, corresponding to required riprap thicknesses of 4.5 to 7 feet. A figure of the calculated riprap sizes utilizing both EAU and FHWA methods is provided as **Figures 4, 5, and 6** for the Rochelle Kaye, Gabby Miller, and Clyde tugs, respectively. Hand calculations verifying the riprap sizes are provided as **Attachment 1**.

Sensitivity Analysis of Riprap Sizes

Based on the sensitivity analysis, it is clear regardless of the selected vessel power or factor of safety, the amount of riprap required to resist propeller wash erosion would be significant at shallower depths. Future armoring design analyses for areas outside of TB4 will continue to assume vessels are operating at 42% power since: (i) this is the EAU recommended assumption for design of bottom protection measures; and (ii) maintenance, construction, and monitoring vessels will likely not be operating at full speed due to the presence of restrictions (three bridges, narrow Canal width). The factor of safety of 1.5 is considered reasonable for the 100%TB4 and 35% RTA1 design. **Table 5** provides the calculations and results of the sensitivity analysis.

Alternative Lining Evaluations

Due to the large riprap sizes that would be required to limit propeller wash scour, alternative lining alternatives were evaluated including marine mattresses, ACBs, and fabric-formed concrete.

Marine mattresses were evaluated based on a permissible shear stress equation developed by Tensar International Corp. (2016). The permissible shear stress was estimated to range from 3.6 to 4.8 psf for marine mattresses with thicknesses ranging from 6 to 24 inches as presented in **Table 6**. The permissible shear stress estimated for marine mattresses is generally less than the bed shear stress calculated at shallower depths (see **Figure 3**), thus utilizing a marine mattress may only be suitable at deeper depths. *The calculated permissible shear stress for marine mattress was significantly less than the critical shear stress cited in the CH2M (2015) technical memorandum of 24.9 psf.*

Permissible shear stress and critical velocity estimates for ACBs were based on manufacturer literature, which was based on hydraulic flume testing, and would not directly replicate the turbulence from propeller wash. The three manufactured products evaluated were as follows:

- **Contech** – The manufacturer estimates the critical velocity of their “Armorflex Class 30S” and “Armorflex Class 40” products to be 15 fps with critical shear stresses ranging from 15 to 35 psf (Koutsourais, 1994).

- **Synthetex -** A manufacturer of ACBs, they cite the shear resistance (i.e., permissible shear stress) of their Open Cell products to range from 26 to 78 psf (Synthetex, 2016).

- **Shoretec** – Shoretec SD 475CC, 600CC, and 900CC products have maximum shear stresses ranging from 5 psf to 10 psf depending on the associated velocity (ACF Environmental, 2016).

Based on the provided data for Contech, Synthetex, and Shoretec products, the permissible shear stress of ACBs is generally larger than the calculated bed shear stress for the tugs at analyzed depths. Thus, the ACBs may be a viable alternative to riprap for lining in RTA1 and the TB4 Pilot Study Area. Associated information for Contech, Synthetex, and Shoretec products is provided as **Appendix 4, 5, and 6**, respectively.

Traditional cast-in place concrete would not be suitable for construction in the Canal without the construction of cofferdams and significant dewatering, however, alternatives such as ACB mats and structural concrete under water would be feasible to construct. The ACB mats would be placed 1 to 2-ft away from the outside edge of proposed and existing bulkheads. Structural concrete tremied into place under water would be used in locations where placement of ACB mats would be difficult due to site restraints (e.g., in areas with space limitations or near bulkhead edges) and what is recommended for use in the TB4 Pilot Study Area. Based on a review of literature, the ACB mats are expected to resist the bed shear stresses due to propeller wash in RTA1 and TB4 Pilot Study Area [ex: the permissible shear stress for Synthetex's Enviromat product is stated as 16 psf (Synthetex, 2016)].

Hydrodynamic Forces

Hydrodynamic characteristics (e.g., bed shear stresses, depth-averaged flow velocities) for forces other than propeller wash were previously analyzed by Baird (2012a) based on existing bathymetric conditions for multiple scenarios as previously discussed and summarized in **Table 7**. The highest peak depth-averaged flow velocity for the hydrodynamic scenarios analyzed in Baird (2012a) was observed at a location approximately halfway between the 3rd and 9th St. Bridges (the Canal appears to slightly narrow down in this area) and estimated to be 1.79 fps, which is approximately one order of magnitude lower than the maximum near-bed velocities induced by propeller wash (see **Tables 2, 3, and 4**). The location of the highest peak bed shear stress was not provided in Baird (2012a), however, it is presumed to be in the main channel of the Canal. In TB4, which is sheltered, the depth-averaged flow velocities and bed shear stresses are significantly less than in the main channel of the Canal and are estimated to range from 0 to 0.1 fps and 0 to 0.025 Pa (= 0 to 0.00052 psf), respectively as presented on Figures F.2 and F.3 in (Baird, 2012a) report. Although the hydrodynamic model may be refined in the future (i.e., updated flushing tunnel flows and proposed surfaces), the results indicate that forces from propeller wash controls the design of the armoring layer.

Ice Scour

Ice thickness was estimated based on the Stefan equation for a presumed "sheltered small river" ice cover condition and the 100-yr recurrence interval for air freezing assuming fresh water conditions. These assumptions are presumed to result in more conservative (larger) ice thicknesses. Based on these assumptions, the 100-yr ice cover thickness was estimated to range from 4 to 8 inches. Since the depth of water in RTA1 after dredging and capping will be significantly greater than 4 to 8 inches, ice scour is not considered a design concern for the cap and will not be evaluated further. A hand calculation is provided as **Attachment 7.**

SUMMARY

The calculation package evaluated the forces that would impact the design and selection of the armoring layer in RTA1 and the TB4 Pilot Study Area including: propeller wash, hydrodynamic forces (i.e., flushing tunnel operations, storm surges), and ice scour. In addition, the feasibility of using traditional armoring (i.e., riprap) and three alternative linings including marine mattresses, ACBs, and fabric-formed concrete was also evaluated.

Based on ice thickness calculations, the formation of ice is estimated to be significantly less than the depth of water (i.e., less than 1-ft vs. 10-ft). Thus, ice scour is not considered to be a design concern for the armoring layer. Bed shear stresses from propeller wash were found to be significant based on the representative tugs analyzed and were calculated to be approximately two orders of magnitude larger than the peak bed shear stress estimated by Baird (2012a) from other hydrodynamic forces (i.e. flushing tunnel, tidal effects) for the scenarios analyzed. Based on the evaluation, the hydraulic design of the armoring layer in RTA1 and TB4 Pilot Study Area would be dictated by propeller wash.

Due to the large bed shear stresses that would occur due to propeller wash at lower depths (i.e., where the top of cap elevation was higher), the thickness of riprap required to limit erosion was estimated to be more than 3 feet. Since an armoring layer this large would have multiple drawbacks (e.g., reduced navigational clearances, difficulty in installation), alternative lining materials are may be more viable for armoring in RTA1 and TB4 Pilot Study Area at shallower depths. For the TB4 Pilot Study Area, it is recommended an ACB mat is selected that has a minimum critical velocity and critical shear stress of 14 fps and 8 psf, respectively, which corresponds with the calculated velocities and shear stresses with a FS of 1.5 anticipated at the top of armor elevations corresponding to -12.7-ft NAVD88.

Of the three alternative lining options evaluated, ACBs are the recommendation for armoring in the TB4 Pilot Study Area and current recommendation in RTA1 due to their ability to limit erosion, constructability, and limited environmental impacts. As the design progresses for RTA1, riprap, fabric-formed concrete, and marine mattresses may be utilized in select locations. Marine mattresses have lower permissible shear stresses relative to ACBs, however, marine mattresses may be suitable at deeper depths. In TB4, structural concrete for underwater applications will be placed near the bulkhead edges.

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TABLES

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Table 1. Description of Tugs Anticipated in RTA1 and TB4 Pilot Study Area

Description	Unit	Tug/Vessel		
		Rochelle Kaye[(1)]	**Gabby Miller[(2)]**	**Clyde[(3)]**
Vessel Length	ft	-	25.5	25
Vessel Breadth	ft	-	14	14
Loaded Draft	ft	6	4	4.5
Rated Power (100%)	hp	700	660	500
Rated Power (100%)[(4)]	KW	522	492	373
Propeller Diameter	ft	3	2.7	2.7
Ducted or non-ducted propeller?		non-ducted	non-ducted	non-ducted
Dual or single propeller?		dual	dual	dual

Notes:

1. Horsepower and draft are based on conversations with the owner of the Rochelle Kaye (Geosyntec, 2016c) and personal knowledge (Geosyntec, 2016d). The vessel has dual propellers, where the propellers were assumed to be 3 feet in diameter and non-ducted, which is similar to other tugs of comparable power and size.
2. Vessel information was obtained based on communications between Geosyntec and Millers Launch (2016).
3. Length, breadth, loaded draft, and rated power were obtained from K-T Marine (2016a). Information on the propeller was obtained based on communications between Geosyntec and K-T Marine (2016b)
4. Conversion from horsepower (hp) to kilowatts (KW)

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Table 2. Propeller Wash Calculations – Rochelle Kaye

PROPELLER WASH CALCULATIONS - ROCHELLE KAYE

INPUTS

Rated Power (horsepower)	700	**Rated Power (KW)**	522
Vessel Draft (ft)	6	**Vessel Draft (m)**	1.83
Propeller Diameter, D (ft)	3	**Propeller Diameter (m)**	0.91
Assumed Power (%)	42%	*corresponds to a rated speed of 75%*	
Design Power, P (horsepower)	294	**Design Power, P (KW)**	219
Density of water ρw (slugs/ft^3)	1.99	**Density of water, ρ (mt/m^3)**	1.025
Unit weight of water, γ_w (pcf)	64		
Unit weight of stone, γ_s (pcf)	150		
Gravity, g (ft/s^2)	32.2		
Factor of Safety	1.5		

COEFFICIENTS

Stability coefficient	1.25
E	0.52
a	-0.28
Shields parameter	0.03

CALCULATIONS

Top of Armor Layer Elevation (ft-NAVD88)	Depth of Water at MLWW (=-2.77-ft NAVD88)	Distance from propeller axis to channel bed, h_p(ft)	Is Equation Valid [0.9 < hp/D <3.0]?[1]		Axial efflux velocity of non-ducted propeller, Vo (ft/s) [EAU, 1996]	Maximum near-bed velocity, max V_{bottom} (ft/s) [EAU, 1996]	Applied Shear Stress, τ_b (psf) [Hayes, et al. 2010]	Applied Shear Stress with Factor of Safety = 1.5, τ_b (psf)	Required Riprap Size, Dreq (ft) [EAU, 1996]	Median Riprap Size, D_{50} (ft) with Factor of Safety = 1.5 [FHWA, 2012]	Riprap Thickness (ft) [EAU, 1996] 2 X D_{req}	Riprap Thickness (ft) [FHWA, 2012] 2 X D_{50}
-12	9.23	4.7	1.6	Yes	30.8	14.1	6.0	9.0	3.0	3.5	5.9	7.0
-12.67	9.9	5.4	1.8	Yes	30.8	13.6	5.3	7.9	2.7	3.1	5.5	6.1
-13	10.23	5.7	1.9	Yes	30.8	13.4	4.9	7.4	2.7	2.9	5.3	5.8
-14	11.23	6.7	2.2	Yes	30.8	12.8	4.2	6.3	2.4	2.5	4.8	4.9
-15	12.23	7.7	2.6	Yes	30.8	12.3	3.7	5.5	2.2	2.1	4.5	4.3
-16	13.23	8.7	2.9	Yes	30.8	11.9	3.2	4.9	2.1	1.9	4.2	3.8
-17	14.23	9.7	3.2	Outside of Range	30.8	11.5	2.9	4.4	2.0	1.7	3.9	3.4
-18	15.23	10.7	3.6	Outside of Range	30.8	11.2	2.6	4.0	1.9	1.5	3.7	3.1
-19	16.23	11.7	3.9	Outside of Range	30.8	10.9	2.4	3.6	1.8	1.4	3.5	2.8
-20	17.23	12.7	4.2	Outside of Range	30.8	10.7	2.2	3.3	1.7	1.3	3.4	2.6

Note:
1. Calculated velocities, shear stresses, and riprap sizes for top of armor layer elevations where the equation is outside of h_p/D range are provided for approximate purposes.

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Table 3. Propeller Wash Calculations – Gabby Miller

PROPELLER WASH CALCULATIONS - GABBY MILLER

INPUTS

Rated Power (horsepower)	660	**Rated Power (KW)**	492
Vessel Draft (ft)	4	**Vessel Draft (m)**	1.22
Propeller Diameter, D (ft)	2.7	**Propeller Diameter (m)**	0.81
Assumed Power (%)	42%	*corresponds to a rated speed of 75%*	
Design Power, P (horsepower)	277	**Design Power, P (KW)**	207
Density of water ρw (slugs/ft^3)	1.99	**Density of water, ρ (mt/m^3)**	1.025
Unit weight of water, γ_w (pcf)	64		
Unit weight of stone, γ_o (pcf)	150		
Gravity, g (ft/s^2)	32.2		
Factor of Safety	1.5		

COEFFICIENTS

Stability coefficient	1.25
E	0.52
a	-0.28
Shields parameter	0.03

CALCULATIONS

Top of Armor Layer Elevation (ft-NAVD88)	Depth of Water at MLWW (=-2.77-ft NAVD88)	Distance from propeller axis to channel bed, h_p(ft)	Is Equation Valid [0.9 < hp/D <3.0]?[1]		Axial efflux velocity of non-ducted propeller, Vo (ft/s) [EAU, 1996]	Maximum near-bed velocity, max V_{bottom} (ft/s) [EAU, 1996]	Applied Shear Stress, τ_b (psf) [Hayes, et al. 2010]	Applied Shear Stress with Factor of Safety = 1.5, τ_b (psf)	Required Riprap Size, Dreq (ft) [EAU, 1996]	Median Riprap Size, D_{50} (ft) with Factor of Safety = 1.5 [FHWA, 2012]	Riprap Thickness (ft) [EAU, 1996] 2 X D_{req}	Riprap Thickness (ft) [FHWA, 2012] 2 X D_{50}
-12	9.23	6.6	2.5	Yes	32.7	13.2	4.3	6.5	2.6	2.5	5.2	5.0
-12.67	9.9	7.2	2.7	Yes	32.7	12.9	3.9	5.9	2.4	2.3	4.9	4.6
-13	10.23	7.6	2.8	Yes	32.7	12.7	3.7	5.6	2.4	2.2	4.8	4.4
-14	11.23	8.6	3.2	Outside of Range	32.7	12.3	3.3	5.0	2.2	1.9	4.5	3.9
-15	12.23	9.6	3.6	Outside of Range	32.7	11.9	3.0	4.4	2.1	1.7	4.2	3.4
-16	13.23	10.6	4.0	Outside of Range	32.7	11.6	2.7	4.0	2.0	1.6	4.0	3.1
-17	14.23	11.6	4.3	Outside of Range	32.7	11.3	2.5	3.7	1.9	1.4	3.8	2.9
-18	15.23	12.6	4.7	Outside of Range	32.7	11.0	2.3	3.4	1.8	1.3	3.6	2.6
-19	16.23	13.6	5.1	Outside of Range	32.7	10.8	2.1	3.1	1.7	1.2	3.4	2.4
-20	17.23	14.6	5.5	Outside of Range	32.7	10.6	1.9	2.9	1.7	1.1	3.3	2.3

Note:
1. Calculated velocities, shear stresses, and riprap sizes for top of armor layer elevations where the equation is outside of h_p/D range are provided for approximate purposes.

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Table 4. Propeller Wash Calculations – Clyde

PROPELLER WASH CALCULATIONS - CLYDE

INPUTS

Rated Power (horsepower)	500	Rated Power (KW)	373
Vessel Draft (ft)	4.5	Vessel Draft (m)	1.37
Propeller Diameter, D (ft)	2.7	Propeller Diameter (m)	0.81
Assumed Power (%)	42%	*corresponds to a rated speed of 75%*	
Design Power, P (horsepower)	210	Design Power, P (KW)	157
Density of water ρw (slugs/ft^3)	1.99	Density of water, ρ (mt/m^3)	1.025
Unit weight of water, γ_w (pcf)	64		
Unit weight of stone, γ_o (pcf)	150		
Gravity, g (ft/s^2)	32.2		
Factor of Safety	1.5		

COEFFICIENTS

Stability coefficient	1.25
E	0.52
a	-0.28
Shields parameter	0.03

CALCULATIONS

Top of Armor Layer Elevation (ft-NAVD88)	Depth of Water at MLWW (=-2.77-ft NAVD88)	Distance from propeller axis to channel bed, h_p(ft)	Is Equation Valid [0.9 < hp/D <3.0]?[1]		Axial efflux velocity of non-ducted propeller, Vo (ft/s) [EAU, 1996]	Maximum near-bed velocity, max V_{bottom} (ft/s) [EAU, 1996]	Applied Shear Stress, τ_b (psf) [Hayes, et al. 2010]	Applied Shear Stress with Factor of Safety = 1.5, τ_b (psf)	Required Riprap Size, Dreq (ft) [EAU, 1996]	Median Riprap Size, D_{50} (ft) with Factor of Safety = 1.5 [FHWA, 2012]	Riprap Thickness (ft) [EAU, 1996] 2 X D_{req}	Riprap Thickness (ft) [FHWA, 2012] 2 X D_{50}
-12	9.23	6.1	2.3	Yes	29.8	12.3	3.9	5.8	2.2	2.3	4.5	4.5
-12.67	9.9	6.7	2.5	Yes	29.8	12.0	3.5	5.2	2.1	2.0	4.2	4.1
-13	10.23	7.1	2.6	Yes	29.8	11.8	3.3	5.0	2.1	1.9	4.1	3.9
-14	11.23	8.1	3.0	Outside of Range	29.8	11.4	2.9	4.4	1.9	1.7	3.8	3.4
-15	12.23	9.1	3.4	Outside of Range	29.8	11.0	2.6	3.9	1.8	1.5	3.6	3.0
-16	13.23	10.1	3.8	Outside of Range	29.8	10.7	2.3	3.5	1.7	1.4	3.4	2.7
-17	14.23	11.1	4.1	Outside of Range	29.8	10.4	2.1	3.2	1.6	1.2	3.2	2.5
-18	15.23	12.1	4.5	Outside of Range	29.8	10.2	2.0	2.9	1.5	1.1	3.1	2.3
-19	16.23	13.1	4.9	Outside of Range	29.8	9.9	1.8	2.7	1.5	1.0	2.9	2.1
-20	17.23	14.1	5.3	Outside of Range	29.8	9.7	1.7	2.5	1.4	1.0	2.8	1.9

Note:

1. Calculated velocities, shear stresses, and riprap sizes for top of armor layer elevations where the equation is outside of h_p/D range are provided for approximate purposes.

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Table 5. Sensitivity Analysis of Key Input Parameters

ROCHELLE KAYE

Top of Armor Layer Elevation (ft-NAVD88)	Underkeel Clearance (ft)	Median Riprap Size, D_{50} (ft) with Factor of Safety [FHWA, 2012]					Required Riprap Size, Dreq (ft) [EAU, 1996]	
		42% Power, FS = 1.2	42% Power, FS = 1.5	100% Power, FS = 1	100% Power, FS = 1.2	100% Power, FS = 1.5	42% Power	100% Power
-12	3.23	2.8	3.5	4.1	5.0	6.2	3.0	5.3
-13	4.23	2.3	2.9	3.4	4.1	5.1	2.7	4.7
-14	5.23	2.0	2.5	2.9	3.5	4.4	2.4	4.3
-15	6.23	1.7	2.1	2.5	3.0	3.8	2.2	4.0
-16	7.23	1.5	1.9	2.2	2.7	3.4	2.1	3.7
-17	8.23	1.4	1.7	2.0	2.4	3.0	2.0	3.5
-18	9.23	1.2	1.5	1.8	2.2	2.7	1.9	3.3
-19	10.23	1.1	1.4	1.7	2.0	2.5	1.8	3.2
-20	11.23	1.0	1.3	1.5	1.8	2.3	1.7	3.0

GABBY MILLER

Top of Armor Layer Elevation (ft-NAVD88)	Underkeel Clearance (ft)	Median Riprap Size, D_{50} (ft) with Factor of Safety [FHWA, 2012]					Required Riprap Size, Dreq (ft) [EAU, 1996]	
		42% Power, FS = 1.2	42% Power, FS = 1.5	100% Power, FS = 1	100% Power, FS = 1.2	100% Power, FS = 1.5	42% Power	100% Power
-12	5.23	2.0	2.5	3.0	3.6	4.5	2.6	4.6
-13	6.23	1.7	2.2	2.6	3.1	3.9	2.4	4.3
-14	7.23	1.5	1.9	2.3	2.7	3.4	2.2	4.0
-15	8.23	1.4	1.7	2.1	2.5	3.1	2.1	3.7
-16	9.23	1.2	1.6	1.9	2.2	2.8	2.0	3.5
-17	10.23	1.1	1.4	1.7	2.0	2.5	1.9	3.4
-18	11.23	1.1	1.3	1.6	1.9	2.3	1.8	3.2
-19	12.23	1.0	1.2	1.4	1.7	2.2	1.7	3.1
-20	13.23	0.9	1.1	1.3	1.6	2.0	1.7	2.9

CLYDE

Top of Armor Layer Elevation (ft-NAVD88)	Underkeel Clearance (ft)	Median Riprap Size, D_{50} (ft) with Factor of Safety [FHWA, 2012]					Required Riprap Size, Dreq (ft) [EAU, 1996]	
		42% Power, FS = 1.2	42% Power, FS = 1.5	100% Power, FS = 1	100% Power, FS = 1.2	100% Power, FS = 1.5	42% Power	100% Power
-12	4.73	1.8	2.3	2.7	3.2	4.0	2.2	4.0
-13	5.73	1.6	1.9	2.3	2.8	3.5	2.1	3.7
-14	6.73	1.4	1.7	2.0	2.4	3.0	1.9	3.4
-15	7.73	1.2	1.5	1.8	2.2	2.7	1.8	3.2
-16	8.73	1.1	1.4	1.6	1.9	2.4	1.7	3.0
-17	9.73	1.0	1.2	1.5	1.8	2.2	1.6	2.9
-18	10.73	0.9	1.1	1.4	1.6	2.0	1.5	2.7
-19	11.73	0.8	1.0	1.2	1.5	1.9	1.5	2.6
-20	12.73	0.8	1.0	1.2	1.4	1.7	1.4	2.5

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Table 6. Marine Mattress Shear Stress Calculations

Mattress Thickness		Permissible Shear Stress(1),(2)
inches	feet	psf
6	0.5	3.6
12	1	4.0
18	1.5	4.4
24	2	4.8

Notes:

1. The specific weight of water and stone were riprap were estimated to be 64 lbs/ft^3 and 150 lbs/ft^3, respectively. The water in the Canal is brackish and was presumed to have a specific weight similar to seawater.

2. Permissible shear stress equation from Tensar International Corp. (2016).

Table 7. Summary of Hydrodynamic Model Results (Baird, 2012a)

Table 5.1 Summary of hydrodynamic model results

Model Run	Tide Condition	Mean Velocity along Centerline (ft/s)	Peak Velocity along Centerline (ft/s)	Mean Shear Stress along Centerline (Pa)	Peak Shear Stress along Centerline (Pa)
Existing Conditions - Neap (with Barges)	Flood	0.09	0.30	< 0.01	0.03
	Ebb	0.10	0.34	< 0.01	0.04
Existing Conditions - Spring (with Barges)	Flood	0.23	0.74	0.03	0.22
	Ebb	0.18	0.60	0.02	0.12
Existing Conditions - Spring (without Barges)	Flood	0.21	0.60	0.03	0.15
	Ebb	0.16	0.48	0.02	0.12
Flushing Tunnel - Spring (with Barges)	Flood	0.21	0.91	0.04	0.47
	Ebb	0.56	1.79	0.16	1.44
Flushing Tunnel - Spring (without Barges)	Flood	0.21	1.02	0.04	0.60
	Ebb	0.52	1.52	0.13	1.29
Hurricane Irene (with Barges)	Flood	0.08	0.25	< 0.01	0.02
	Ebb	0.22	0.67	0.03	0.22

Notes:

1. The calculations were based on existing bathymetric conditions at the time and do not represent proposed capping surfaces. Flushing tunnel flows used in Baird's analysis were based on average flushing tunnel flows of 215 million gallons per day (MGD), which is the future target flow rate as stated in the Gowanus Canal Waterbody/Watershed Facility Plan Report (NYCDEP, 2008). If the analysis is updated in the future, it would need to account for variation in flushing tunnel flow rates at low and high tide of 175 MGD and 252 MGD, respectively (NYCDEP, 2008). The most recent discharge rates and operating conditions would need to be obtained from NYCDEP.

2. One (1) pound per square foot (psf) is equal to approximately 47.9 Pascals (Pa). Thus, a bed shear stress of 1.44 Pa is equal to 0.03 psf.

FIGURES

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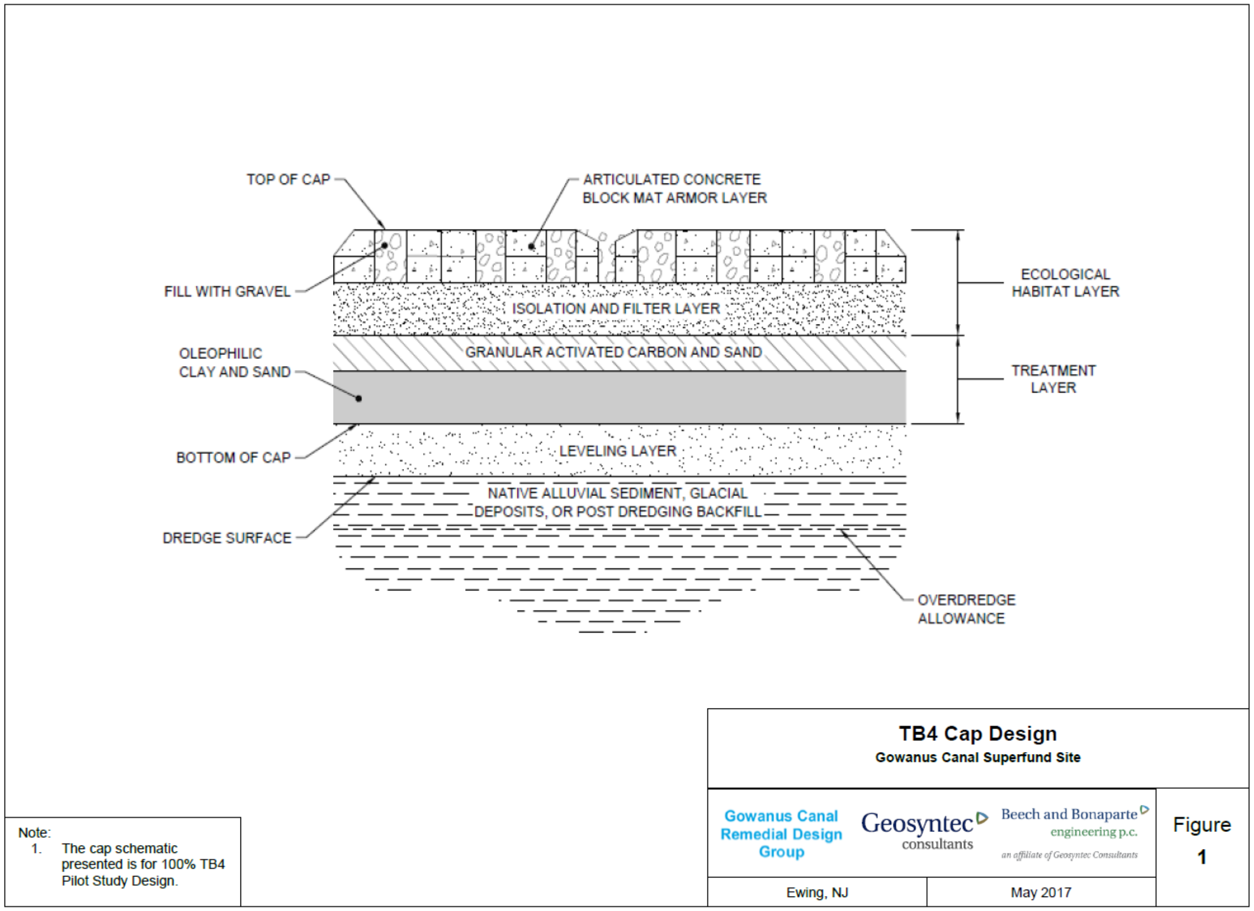

Figure 1. Schematic of the Proposed Cap Design in TB4 Pilot Study Area

(Note: The Cap Design in RTA1 is anticipated to be similar)

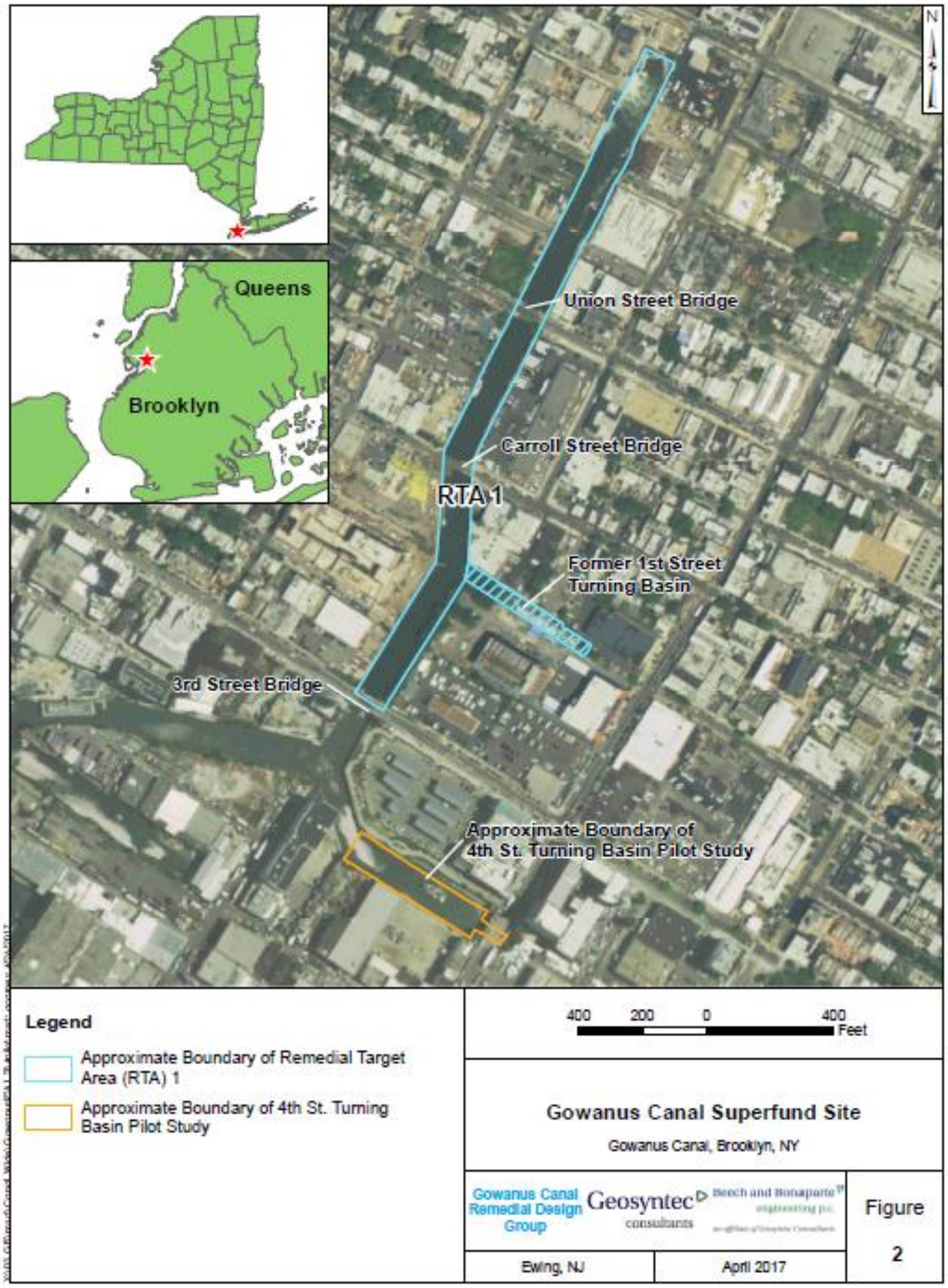

Figure 2. RTA1 and TB4 Pilot Study Area *(Note: The Limits of Capping differ in TB4 from the total area of the TB4 Pilot Study at the western limits and beneath the 3rd Ave. Bridge)*

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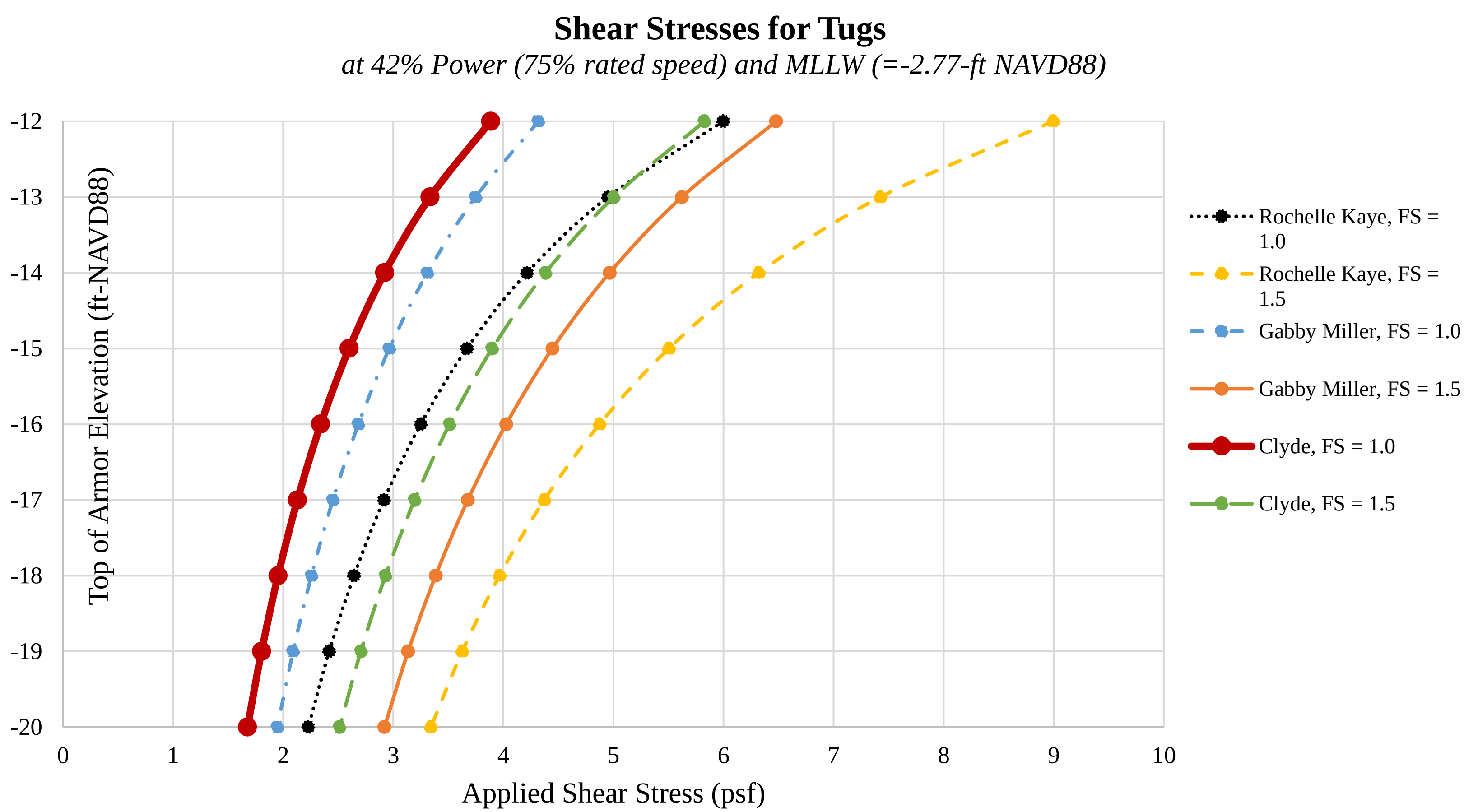

Figure 3. Bed Shear Stresses for Tugs

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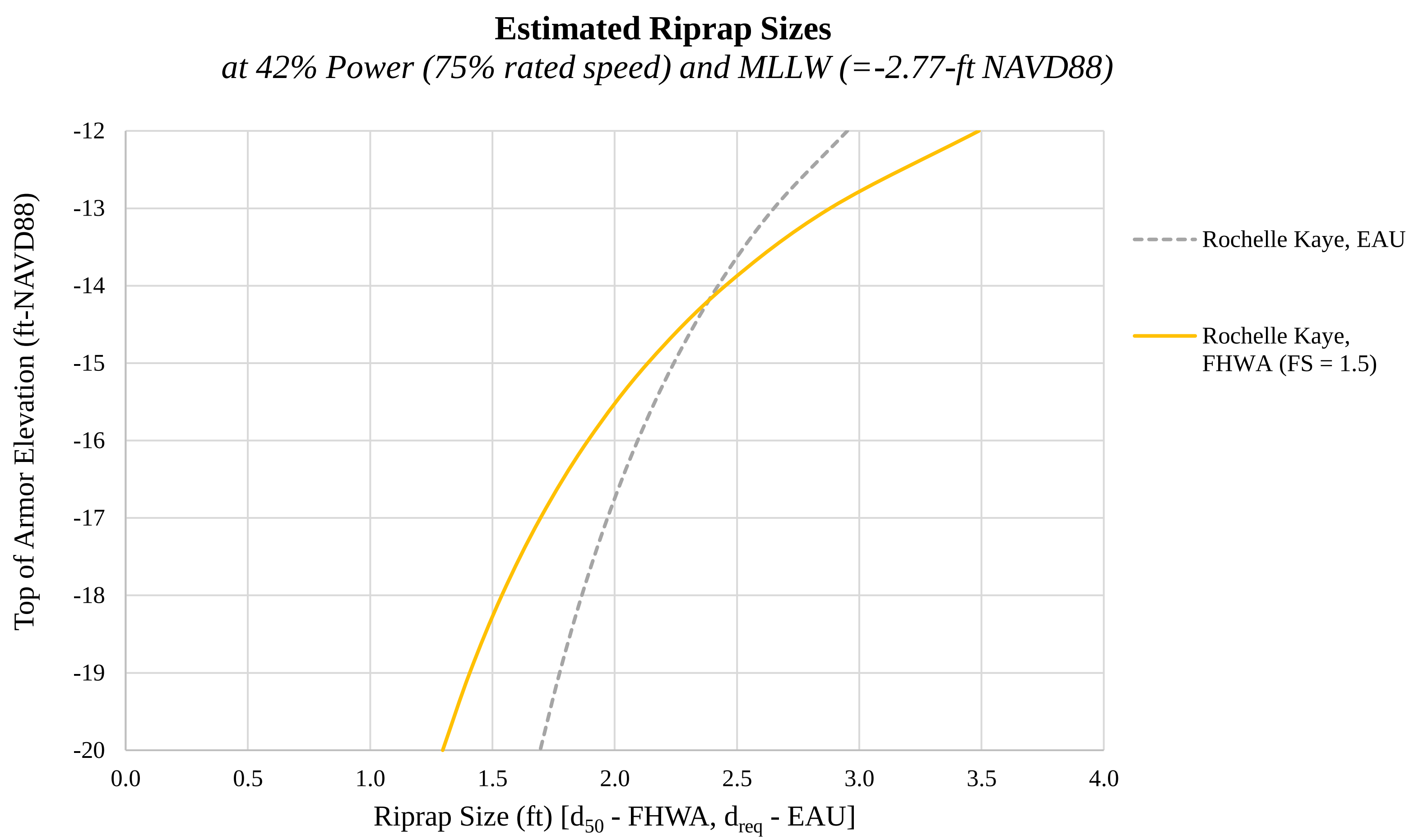

Figure 4. Estimated Riprap Sizes – Rochelle Kaye

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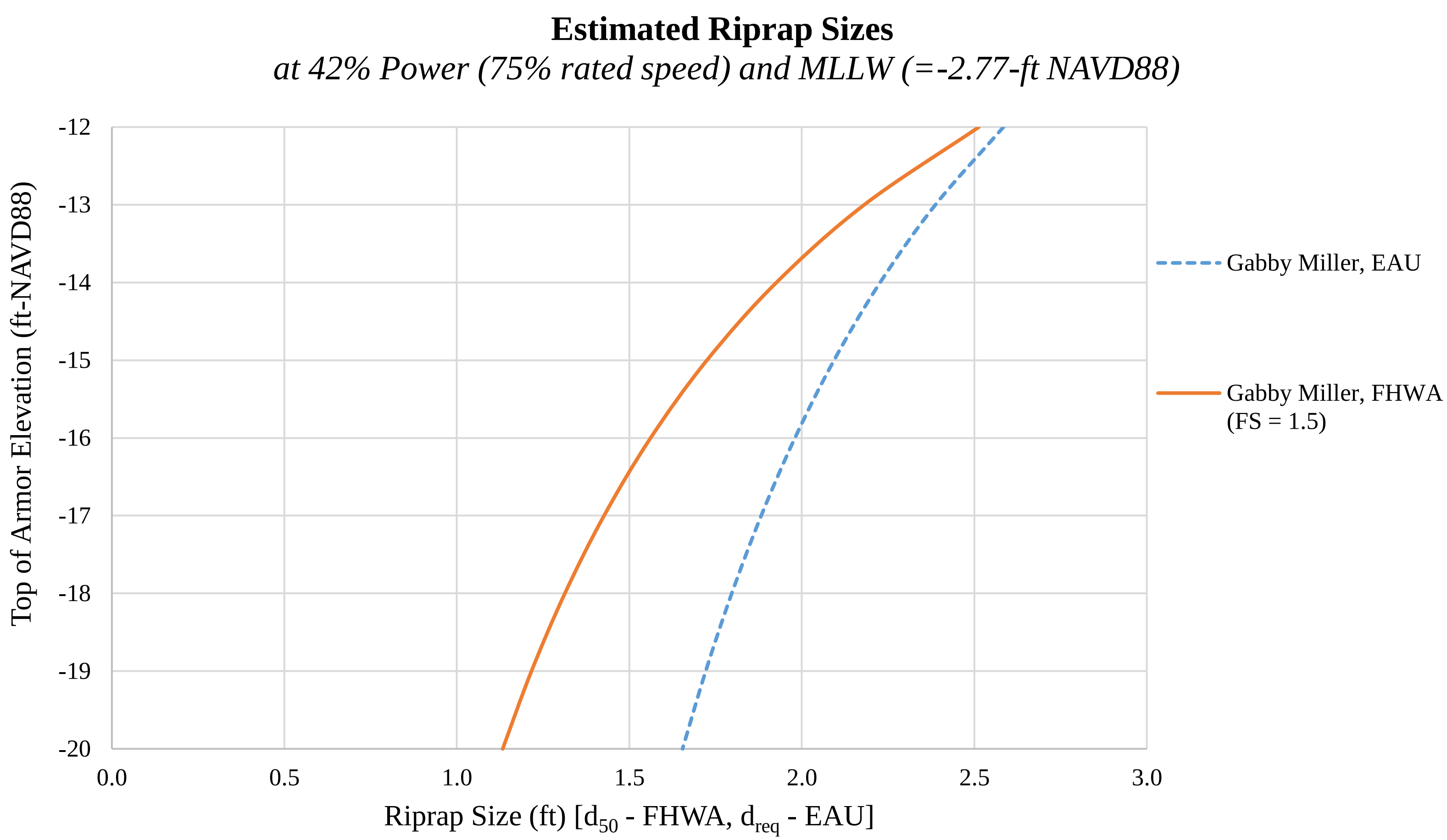

Figure 5. Estimated Riprap Sizes – Gabby Miller

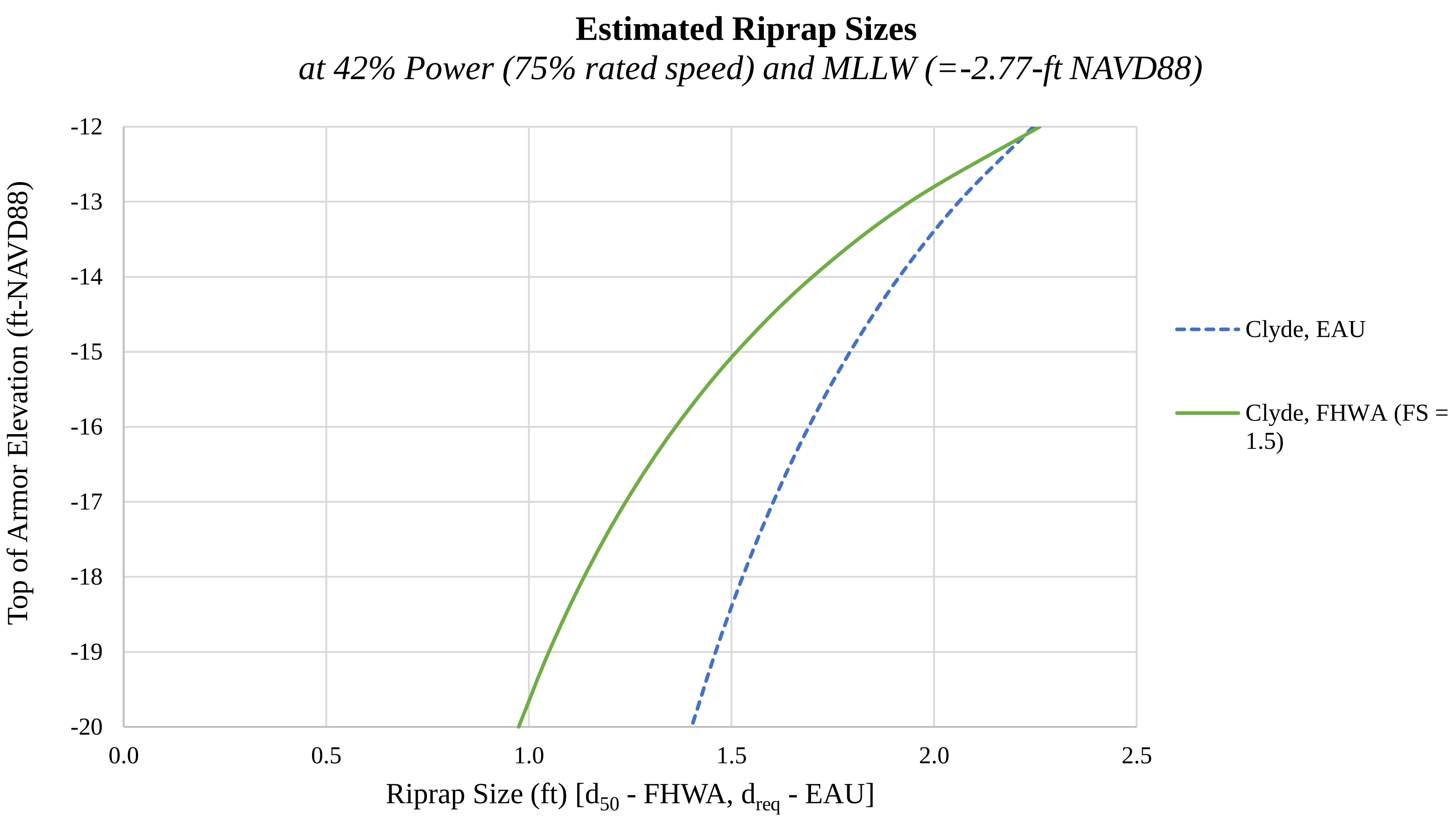

Figure 6. Estimated Riprap Sizes – Clyde

ATTACHMENTS

ATTACHMENT 1 – HAND CALCULATIONS FOR PROPELLER WASH NEAR-BED VELOCITIES AND BED SHEAR STRESSES

CP: SS Date: 5/19/2017 APC: GDN Date: 5/19/2017 CC: MWS Date: 5/19/2017

Client: RD Group Project: Gowanus Canal Superfund Site Project No: HPH106A

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Geosyntec consultants

Written by: M. Schillinger Date: 23/11/16 Reviewed by: SHAURYA SOOD Date: 29/11/16

Client: RD Group Project: Gowanus Project/Proposal No. HPH106A Task No. 40.04

EAU method

EXAMPLE CALCULATION FOR GABBY MILLER

JET PROPELLER VELOCITY (V_o)

$$V_o\ (m/s) = C_p \left(\frac{P}{\rho_w D^2}\right)^{1/3}$$

where:

$C_p = 1.48$ for non-ducted propellers

$P = 660$ hp @ 100% power

$660\ hp \times \frac{0.746\ kW}{hp} = 492\ kW$ @ 100% power

at 75% rated speed or 42% power $= 492\ kW \times 0.42 = 207\ kW$

$\rho_w = 1.025$ metric tons$/m^3$ for seawater

$D = 2.67\ feet \times \frac{1\ meter}{3.28\ feet} = 0.81$ meters

Thus

$$V_o\ (m/s) = 1.48 \left(\frac{207\ kW}{1.025\ \frac{metric\ tons}{m^3} \times (0.81\ m)^2}\right)^{1/3} = \sim 10\ m/sec$$

@ 42% power

$$V_o = 10\ m/sec \times \frac{3.28\ feet}{1\ meter} = \sim 33\ feet/sec$$

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Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

Geosyntec consultants — Written by: M. Schillinger Date: 23/11/16 Reviewed by: SHAURYA SOOD Date: 29/11/16
Client: RD Group Project: Gowanus Project/Proposal No. HPH106A Task No. 40.04

EAU method

MAXIMUM NEAR-BED VELOCITY (MAX V_{bottom}) at Top of Cap/Armor
Elevation = −12 ft

$$\text{MAX } V_{bottom} = V_0 \, E \left(\frac{h_p}{D}\right)^a$$

where:

V_0: 10 m/sec or 32.7 fps

E: 0.52 for twin-screw with twin rudders
* assumed to have rudders

$D = 2.67 \text{ ft} \times \frac{1 \text{ meter}}{3.28 \text{ feet}} = 0.81$ meters (propeller diameter)

a = −0.28 for twin-screw

h_p (distance from propeller axis to channel bed)
= Depth of water − Loaded Draft + ½ × D

where Depth of water at MLLW = Δ(MLLW and Top of Cap)
= −2.77 ft NAVD88 − (−12 ft) = 9.23 ft
Loaded Draft = 4 feet

$h_p = 9.23 \text{ ft} - 4 \text{ ft} + \tfrac{1}{2}(2.67 \text{ ft}) = 6.6$ feet

$$\text{Max } V_{bottom} = 32.7 \text{ fps} \times 0.52 \left(\frac{6.6 \text{ feet}}{2.7 \text{ feet}}\right)^{-0.28} = \underline{13.2 \text{ fps}}$$

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Written by: M. Schillinger Date: 29 / 11 / 16 Reviewed by: SHAURYA SOOD Date: 29 / 11 / 16

Client: RD Group Project: Gowanus Canal Project/Proposal No.: HPH106A Task No.: 40.04

SHEAR STRESS ON THE BED HAYES, ET. AL (2010) METHOD

MLLW = -2.77 - FT NAVD88

ASSUMED TOP OF ARMOR LAYER = -12- FT NAVD88

$\tau_B (psf) = \frac{1}{2} C_f \rho_w V_0^2$

where $C_f = 0.01 \left({}^{D}/_{h_p} \right) = 0.01 \left(\frac{2.67 \, ft}{6.6 \, ft} \right) = 0.004$

$\tau_B = \frac{1}{2}(0.004)(1.99 \; {}^{slugs}/_{cf})(32.7 \; fps)^2 = \underline{\underline{4.3 \; psf}}$

ATTACHMENT 2 – HAND CALCULATIONS FOR RIPRAP SIZING

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Client: RD Group Project: Gowanus Canal Superfund Site Project No: HPH106A

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Written by: M Schillinger Date: 23/11/16 Reviewed by: SHAURYA SOOD Date: 29/11/16

Client: RD Group Project: Gowanus Project/Proposal No. HPH106A Task No. 40.04

RIPRAP SIZING (EAU METHOD)

$$d_{req} \geq \frac{(\text{Max } V_{\text{bottom}})^2}{B^2 g} \frac{\rho_w}{\rho_s - \rho_w}$$

$$\geq \frac{(\text{Max } V_{\text{bottom}})^2}{B^2 g} \frac{\gamma_w}{\gamma_s - \gamma_w} = \frac{(13.2 \text{ ft/s})^2}{32.2 \frac{ft}{s^2} (1.25)^2} \times \frac{64 \text{ pcf}}{150 - 64 \text{ pcf}}$$

1.25 for stern screws with central rudders

d_{req} (ft) $\geq$ 2.6 ft

Thus if 2.6 ft is the d_{50}, the thickness would be 5.2 ft

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Written by: M. Schillinger Date: 23/11/16 Reviewed by: SHAURYA SOOD Date: 29/11/16
Client: RD Group Project: Gowanus Project/Proposal No. HPH106A Task No. 40.04

RIPRAP SIZE BASED ON SHEAR STRESS (FHWA, 2012)

$$d_{50} = \frac{\tau_b}{k_s g (\rho_s - \rho_w)} = \frac{(FS)\tau_B}{k_s (\gamma_s - \gamma_w)}$$

└ Shields parameter

$\tau_B = 4.3$ psf with 1.5 factor of Safety $= 6.5$ psf

$$d_{50} = \frac{6.5 \text{ psf}}{0.03 (150 \text{ pcf} - 64 \text{ pcf})} = 2.5 \text{ feet}$$

Thus thickness of $2 \times d_{50} = 2.5$ feet $\times 2 = 5.0$ feet

ATTACHMENT 3 – HAND CALCULATIONS FOR MARINE MATTRESS PERMISSIBLE SHEAR STRESSES

CP: **SS** Date: **5/19/2017** APC: **GDN** Date: **5/19/2017** CC: **MWS** Date: **5/19/2017**

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Geosyntec consultants

Written by: MARK SCHLLINGER Date: 23 / 11 / 16 Reviewed by: SHAURYA SOOD Date: 29 / 11 / 16

Client: RD Group Project: Gowanus Canal Project/Proposal No. HPH106A Task No. 40.04

Permissible Shear Stresses for Marine Mattresses

Example calculation for 12" mattress

$\tau_p = 0.0091(\gamma_s - \gamma_w)(MT + c)$ from Kilgore and Cotton, 2005

where

$\gamma_s = 150$ lb/ft³ density of riprap

$\gamma_w = 64$ lb/ft³ density of seawater

$MT = 12''$ $(= 1')$

$C = 4.07$ (constant for English units)

$\tau_p\,(psf) = 0.0091(150 - 64 \text{ lb/ft}^3)(1' + 4.07)$

$= 3.97$ psf or ≈ 4 psf

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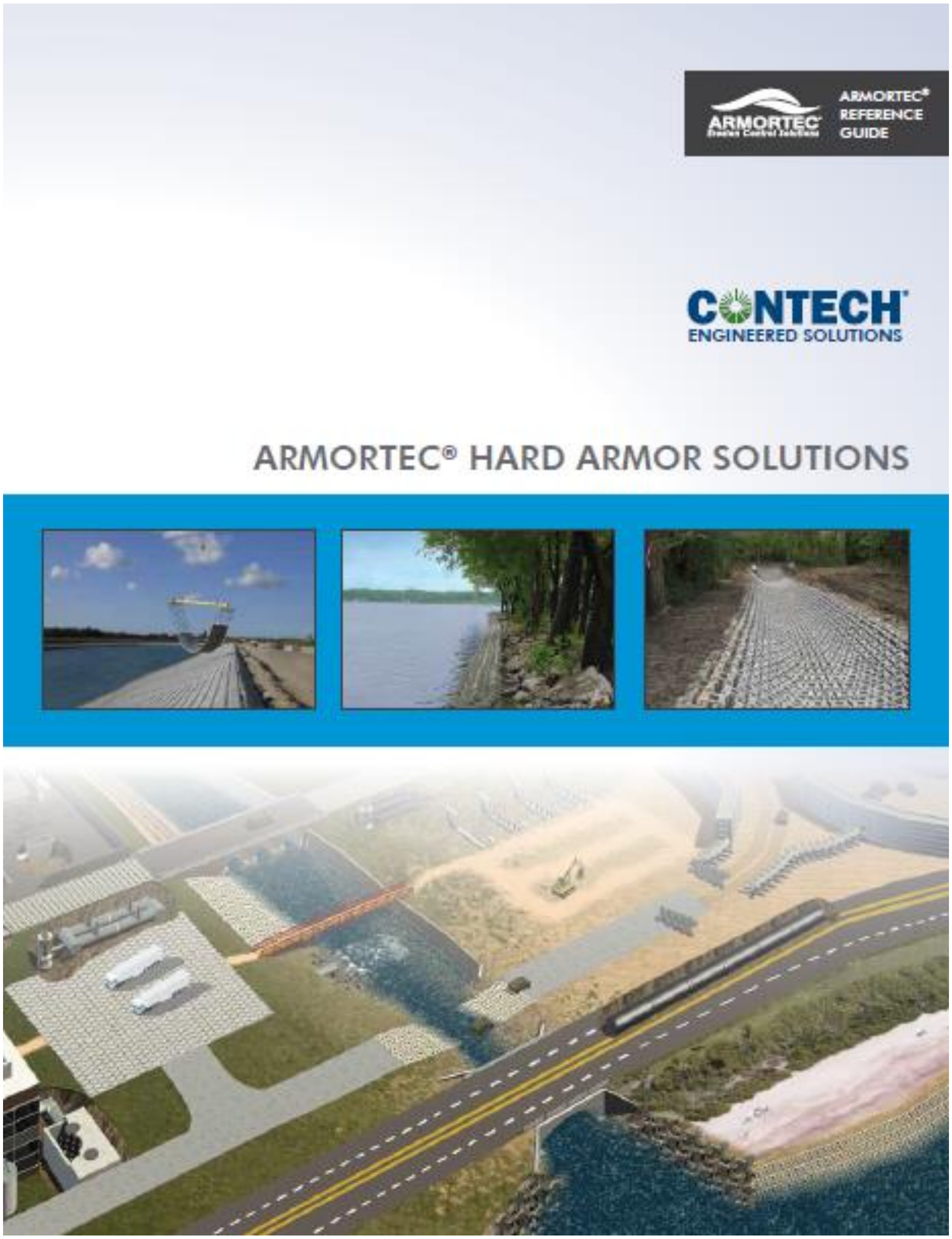
Client:	**RD Group**	Project:	**Gowanus Canal Superfund Site**	Project No:	**HPH106A**

ATTACHMENT 4 – ARMORTEC PRODUCT LITERATURE FROM CONTECH ENGINEERED SOLUTIONS (2016)

CP:	**SS**	Date:	**5/19/2017**	APC:	**GDN**	Date:	**5/19/2017**	CC:	**MWS**	Date:	**5/19/2017**

Client:	**RD Group**	Project:	**Gowanus Canal Superfund Site**	Project No:	**HPH106A**



MATTED SOLUTIONS

ARMORFLEX® ARTICULATING CONCRETE BLOCKS

OPEN CELL BLOCK DESIGN ALLOWS FOR REVEGETATION

CLOSED CELL BLOCK DESIGN ALLOWS FOR HEAVY LOADING

BOTH BLOCKS READILY ADAPT TO COMPLEX SITE GEOMETRIES

BLOCK OPTIONS

Open-Cell Block

Closed-Cell Block

Tapered-Cell Block

Block and a Half®

ARMORFLEX UNIT SPECIFICATION

Block Class	Open/Closed Cell	Nominal Thickness	Gross Area (sf)	Block Weight (lbs)	Open Area %
30-S	Open	4.75	0.98	33-35	20
50-S	Open	6.00	0.98	42-45	20
40	Open	4.75	1.77	59-64	20
50	Open	6.00	1.77	76-82	20
70	Open	8.50	1.77	108-117	20
40-L	Open	4.75	2.88	97-105	20
70-L	Open	8.50	2.88	174-188	20
45-S	Closed	4.75	0.98	39-42	10
55-S	Closed	6.00	0.98	50-54	10
45	Closed	4.75	1.77	71-77	10
55	Closed	6.00	1.77	91-98	10
85	Closed	8.50	1.77	136-146	10
45-L	Closed	4.75	2.88	109-118	10
85-L	Closed	8.50	2.88	207-223	10
High Velocity Application Block Classes					
40-T	Open	4.75	1.77	58-63	20
50-T	Open	6.00	1.77	75-81	20
70-T	Open	8.50	1.77	116-124	20

4 | WWW.CONTECHES.COM/HARD-ARMOR

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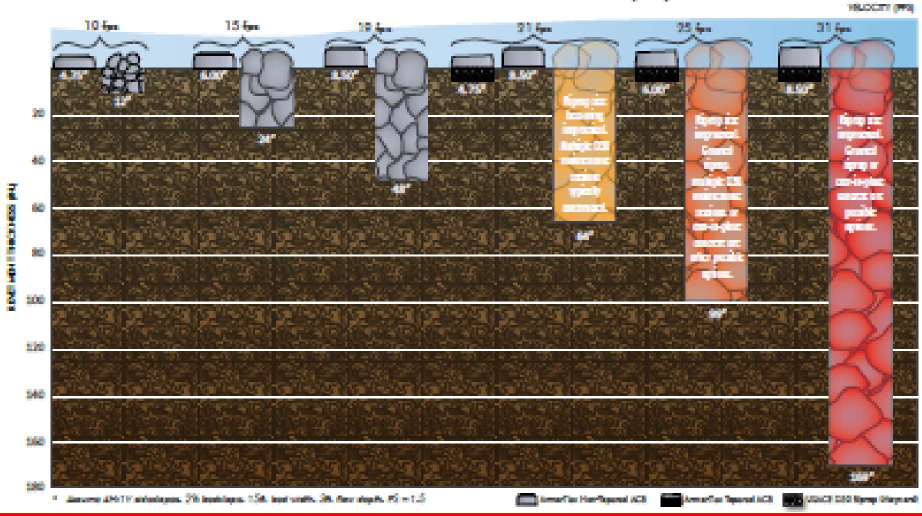
MATTED SOLUTIONS

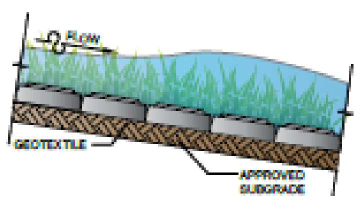
ARMORFLEX® DESIGN CONSIDERATIONS

5 –

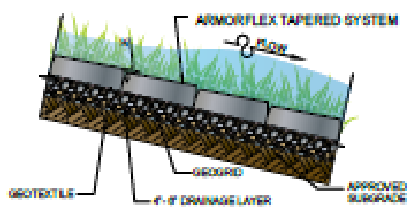
SIZING



TYPICAL CROSS SECTIONS (not to scale)



Standard Cross Section



Tapered Series - Cross Section

REFERENCES AND STANDARDS

- National Concrete Masonry Association (2010), "Design Manual for Articulating Block (ACB) Revetment Systems", NCMA Publication TR 220A
- ASTM D 7276 – Standard Guide for Analysis and Interpretation of Test Data for ACB Revetment Systems in Open Channel Flow
- ASTM D 7277 – Standard Test Method for Performance Testing for ACB Revetment Systems for Hydraulic Stability in Open Channel
- ASTM D 6684 – Standard Specification for Materials and Manufacture of Articulating Concrete Block (ACB) Revetment Systems
- ASTM D 6884 – Standard Practice for Installation of Articulating Concrete Block (ACB) Revetment Systems
- FHWA Hydraulic Engineering Circular NO. 23: Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance – Third Edition, Volume II, Design Guideline 8.
- USDOT Federal Highway Administration Hydraulic Engineering Circular NO. 15, Third Edition (2005) "Design of Roadside Channels with Flexible Linings" National Highway Institute.
- Julien, Pierre Y. (2010) "Erosion and Sedimentation", 2nd Edition, Cambridge University Press

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Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

ARMORTEC

Concrete Erosion Control Systems

CASE HISTORY

A study of articulated concrete blocks designed to protect embankment dams

By Michael Koutsourais, P.E.

Abstract

In recent years articulated concrete blocks (ACBs) have provided innovative and cost-effective solutions for embankment dam protection. Many older, outdated dams have been rehabilitated to meet higher regulatory standards and/or to accommodate current estimates of a probable maximum flood.

A cost-effective method of upgrading many of these dams is to allow overtopping and to use revegetated concrete blocks designed for hydraulic stability to protect the downstream face of the dam. Armoring the embankment helps prevent erosion caused by steep-gradient, high-velocity flow.

This article presents several case studies where ACB systems are used to prevent erosion of the downstream face of a dam during overtopping flow.

Introduction

An ACB system is a matrix of individual concrete blocks assembled to form a large mat. ACBs may be hand-placed on site or threaded with cable into prefabricated mattresses up to 45 square meters and placed with a spreader-bar attached to a large crane. The concrete blocks range from 100 millimeters to 225 millimeters in height, 15 kilograms to 75 kilograms in weight, 0.09 square meters to 0.18 square meters in gross area and 4 percent to 25 percent in open area.

Revetment cables are made of polyester or galvanized steel and depending upon the block size and mat length, are 3.2 millimeters to 12.7 millimeters in diameter. Cables typically run longitudinally but also may run laterally if no inter-block restraint is otherwise provided.

Of the many erosion-protection systems, structural-reinforced concrete, roller-compacted concrete or soil cement traditionally are used to achieve the desired performance level and stability under the high-stress applications associated with embankment overtopping flow conditions. However, in addition to providing a cost-effective and hydraulically stable armoring

Table 1. Results of flume study conducted by Simons, Li and Associates Inc.[1,2]

Block Type	Critical Velocity m/s [ft/s]	Critical Shear Stress kPa [psf]
Dycel 100	2 (7)	< 0.35 (7)
[illegible]	[illegible]	[illegible]
Armorflex Class 30S	4.5 (15)	0.72 (15)
Armorflex Class 40	4.5 (15)	1.63 (35)**
Concrete construction blocks	5 (17)	0.96 (20)**
Wedge-shaped blocks	5 (17)	1.20 (25)**

* All blocks were tested under bare (unvegetated) conditions.
** Shear stress at maximum flume capacity; no failure noted. Use of these values will result in conservative design.

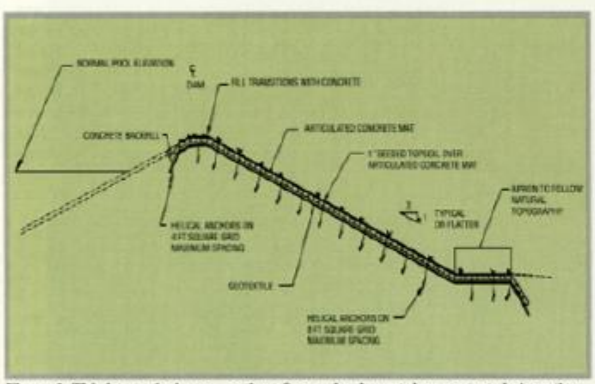

Figure 1. This is a typical cross-section of an embankment dam protected along the crest and downstream slope with an ACB system.

Figure 2. Articulated concrete blocks protect dams at the Blue Ridge Parkway.[2]

Figure 3. Articulated concrete blocks protect the Wilkerson Lake Dam in Georgia.

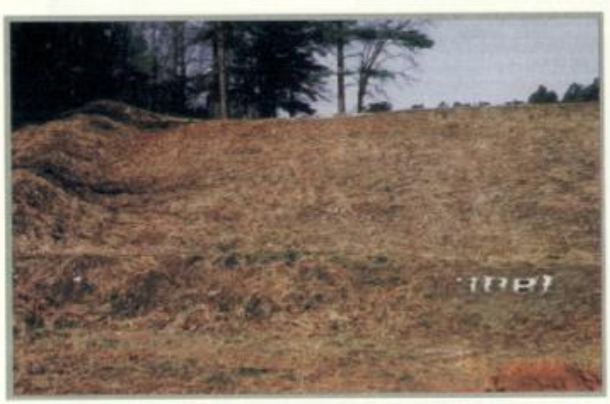

Figure 4. The Wilkerson Lake Dam was constructed in the spring of 1992.

solution, ACBs also provide a solution that can be revegetated, thus combining environmental attractiveness with durability and hydraulic efficiency.

Other positive features of ACBs are their ability to release hydrostatic pressures through the cell openings and through the periphery of the individual blocks and their ability to accommodate minor subgrade changes caused by settlement, frost heave and surface slumping.

Research

The former Soviet Union has been using concrete blocks for overtopping protection of earth dams since the mid-1970s. The concrete blocks have been successfully protecting earth dams ranging from 20 meters to 60 meters high for more than a decade.[1] It was only a few years ago, however, that the first North American dams, located in the National Park Service's Blue Ridge Parkway in North Carolina, were protected using an ACB system. **Figure 1** shows a typical cross-section of an embankment dam protected along the crest and downstream slope with an ACB system.

During the 1980s, ACBs were developed as a viable solution in preventing erosion of the downstream face of a dam during overtopping. United States and Great Britain investigators conducted hydraulic test programs to determine the reliability of various ACB systems under these high-stress conditions.

One of the testing programs (performed by Simons, Li & Associates Inc. [SLA], Fort Collins, Colo., and funded by several government agencies), subjected the products to increasingly severe overtopping flows until failure of the system occurred or maximum hydraulic capacity of the facility was reached. The maximum unit discharge of the testing facility was 2.3 cm/s/m associated with 1.22 meters of overtopping.

Table 1 shows that many articulated concrete block systems successfully resisted full flume capacity with velocities up to 6 meters per second[2,3] and shear stresses up to 1.68 kPa (35 psf).[1,2,3] Note that SLA conducted the study of ACB systems under bare or unvegetated conditions.

Another research study, performed by the Construction Industry Research and Information Association (CIRIA), in Great Britain, was conducted to determine the hydraulic stability of erosion-protection systems once a vegetative stand is established. The CIRIA research indicates that several ACB systems will resist up to 8 meters per second[4] once vegetation is established and that the hydraulic stability of these systems is independent of flow duration.[4] The results of the studies also indicate that ACB systems can achieve a high degree of stability under high-velocity, high-stress conditions.

Case studies

The following case studies feature eight embankment dams that have been designed to accommodate overtopping flow by protecting the downstream face with a revegetated ACB system.

Figure 5. Articulated concrete blocks protect the Strahl Lake Dam.

Blue Ridge Parkway Dams[5]

In the fall of 1990, the first North American installation of an ACB system as embankment overtopping protection was performed on three small dams located in the National Park Service's Blue Ridge Parkway in North Carolina (**Figure 2:** Bass Lake Dam, Price Lake Dam and Trout).

Calculated overtopping heights for the design flood range from 0.60 meters at Bass Lake dam to 1.25 meters at Price Lake Dam. The dams range from 8.54 meters to 12.20 meters high at the maximum sections. Calculated maximum velocities occurred at the embankment toes and ranged from 6.7 meters per second at Bass Lake dam to 7.9 meters per second at Price Lake Dam.

The decision to use an ACB system was based on its successful performance under flow velocities of up to 7.9 meters per second during full-scale tests in Great Britain and because of the anticipated lower cost when compared to other types of overtopping protection.

Because appropriate design guidelines were unavailable for this system, GEI used the results of the embankment overtopping studies performed by SLA and by CIRIA and made hydraulic comparisons of the flow conditions in the research study with the design conditions at the Blue Ridge Parkway Dams, and performance comparisons of the various ACB systems tested.

The downstream face of the dams are at a slope of 3(H):1(V). The installed block weighs 27 kilograms over a 0.12 square-meter area, and provides an open area of approximately 20 percent to allow for revegetation through the cell openings. Helical and duckbill anchors on a 2.44 meters by 2.44 meters spacing provide additional resistance to the overtopping flow. A woven geotextile with an apparent opening size of 0.42 millimeters and at least 10 percent open area provides adequate retention and long-term permeability. Total material and installation cost of the ACB system including geotextile and anchors was approximately $65 per square meter.

Wilkerson Lake and Run Dams

The Savannah District Corps of Engineers recently was faced with the problem of upgrading Wilkerson Lake, **Figure 3**, and the three Sandy Run Dams near Augusta, Ga., to accommodate current estimates of the probable maximum flood (PMF). The solution was to allow limited overtopping of the dam and to design an ACB system to protect the crest and downstream face while providing an attractive revegetative surface. **Figure 4** shows the revegetated ACB system one year after placement on the Wilkerson Lake Dam.

The Wilkerson Lake dam was constructed in the spring of 1992, and the three Sandy Run dams are scheduled to be complete in 1994. The dams, classified as low hazard, are approximately 5 meters high. Calculated velocities from the design storm of 0.46 meters of overtopping are approximately 4.9 meters per second with shear stresses approximately 0.7 kPa (15 psf).

By comparing the design hydraulic conditions with conditions successfully resisted in the previous mentioned research studies, a block weight of 22 kilograms over an area of 0.09 square meters was determined to provide the necessary resistance against the overtopping flows on the downstream 3(H):1(V) slope of the Wilkerson and the Sandy Run Dams. Additional resistance is provided by installing helical anchors on a 2.44 meters by 2.44 meters pattern throughout the downstream face of the dam A woven geotextile filter with a 0.42 millimeter apparent opening size and an open area of at least 10 percent was installed beneath the system.

Strahl Lake Dam

The Strahl Lake Dam, **Figure 5**, regulated by the Indiana Department of Natural Resources, is approximately 50 years old and is located in the Brown County State Park near Indianapolis. Rehabilitation of the dam was performed in 1993 and includes an ACB system as the downstream face protection designed by Fink, Roberts, & Petrie, Indianapolis. The dam is approximately 7.3 meters high and is classified as high hazard. Therefore, it was critical that the embankment protection system of the dam be designed to resist the 60 percent probable maximum flood (PMF) and provide an attractive vegetative surface for the park visitors.

Fink, Roberts, & Petrie used the safety factor technique[7] developed by SLA to size the articulated concrete block based on the site-specific hydraulic conditions. This technique originally was developed to size riprap in open-channel flow conditions.

Figure 6 is a schematic of the forces that affect block stability and are used in the safety factor methodology. The forces of lift and drag act to destabilize each individual block or cause the most vulnerable block within the matrix to lose intimate contact with the subgrade, the condition of incipient failure.[1]

The direction of the lift force, F_L, is perpendicular to the bottom surface of the block and acts to displace the block upward. The direction of the drag force, F_D, is consistent with the direction of flow and acts to overturn the block about its downstream edge. An additional form drag force, F_D, can be calculated and incorporated into the safety factor analysis to account for the possible occurrence of differential pressures acting behind the block as well as to account for additional forces that occur on a block that protrudes above adjacent upstream blocks.

Calculated velocity for the design storm of 0.7 meters of overtopping is approximately 4.9 meters per second with a shear stress of 0.9 kPa (19 psf). The designed block has a weight of 47 kilograms an area of approximately 0.17 square meters and was installed on a 3(H):1(V) downstream slope face. The geotextile filter is a woven monofilament/multifilament combination which has an apparent opening size of 0.25 millimeters and an open area of at least 10 percent.

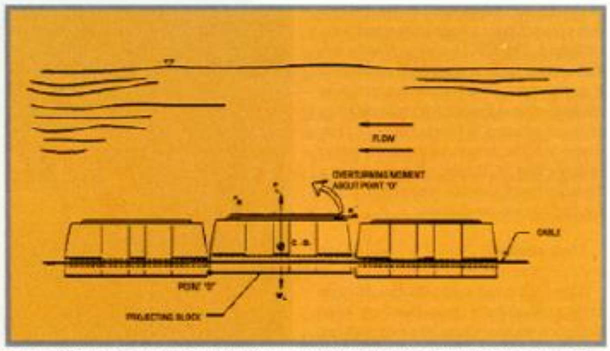

Figure 6. This is a schematic of the forces that affect block stability and are used in the safety factor methodology.

Summary

There are many outdated dams in the United States that require rehabilitation. A cost-effective, hydraulically stable and environmentally attractive method of dam upgrading is to allow overtopping and to protect the downstream face of the dam using a revegetated ACB system.

Designers of these dams must feel comfortable with the selection of block type, weight and dimensions of this type of protection system to ensure design performance. The hydraulic testing performed during the 1980s has provided performance information of various block systems under ideal conditions and has shown that ACBs can achieve a high degree of stability under high-velocity, high-stress conditions.

Analytical techniques are available to extrapolate the results of the hydraulic tests to geometrically similar blocks of different sizes and/or weights and to determine their performance under hydraulic conditions which are less than ideal. Several case histories have been presented to provide an historical use of ACBs as embankment dam protection.

References

[1]Clopper, P.E., "Protecting Embankment Dams with Concrete Block Systems," published in the Hydro Review, Vol. X, No. 2, April 1991

[2]Clopper, P.E. and Y. H. Chen, "Minimizing Embankment Damage During Overtopping Flow," final report, Simons, Li & Associates Inc., Fort Collins, Colo., prepared for the Federal Highway Administration, Report No. FHWA-RD-88-181, Washington, D.C., November 1988

[3]Clopper, P.E. "Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow," Final Report, Simons, Li & Associates Inc., Fort Collins, Colo., prepared for the Federal Highway Administration, U.S. Bureau of Reclamation, Soil Conservation Service, and Tennessee Valley Authority, Report No. FHWA-RD-89-199, July 1989

[4]Construction Industry Research and Information Association (CIRIA), "Design of Reinforced Grass Waterways," Report No. 116, 6 Storey's Gate, London, England SW1P 3AU 1987

[5]Wooten, R. Lee, Powledge, George R., and Whiteside, Stephen L., "Dams Going Safely Over the Top," Civil Engineering, ASCE, January 1992, pp. 52-54

Acknowledgements

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About the author *Michael Koutsourais, P.E., is the Northeast regional engineer for the Nicolon/Mirafi Group, a division of Nicolon Corp., Norcross, Ga.*

CP: **SS** Date: **5/19/2017** APC: **GDN** Date: **5/19/2017** CC: **MWS** Date: **5/19/2017**

Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

ATTACHMENT 5 – ACB PRODUCT LITERATURE FROM SYNTHETEX, LLC (2016)

Section ____________

EROSION CONTROL LINING SYSTEM SPECIFICATION
ARTICULATING BLOCK AB400LL FABRIC FORMED CONCRETE

PART 1.0: GENERAL

1.1 Scope of Work

The work shall consist of furnish all labor, materials, equipment, and incidentals required and perform all operations in connection with the installation of the fabric formed concrete erosion control lining systems in accordance with the lines, grades, design, and dimensions shown on the Contract Drawings and as specified herein. If the contractor is inexperienced, then the fabric formed concrete manufacturer's representative shall provide on-site technical assistance at the beginning of the installation for a length of time the contractor is sufficiently experienced to complete the remaining installation.

1.2.1 Description

The work shall consist of installing a reinforced concrete lining by positioning specially woven, double-layer synthetic forms on the surface to be protected and filling them with a pumpable fine aggregate concrete (structural grout) in such a manner as to form a stable lining of required thickness, weight and configuration.

1.3 Referenced Documents

1.3.1 American Society for Testing and Materials (ASTM)

ASTM C 31	Standard Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C 33	Standard Specification for Concrete Aggregates
ASTM C 94	Standard Specification for Ready-Mixed Concrete
ASTM C 109	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or [50-mm] Cube Specimens)
ASTM C 150	Standard Specification for Portland Cement
ASTM C 260	Standard Specification for Air-Entraining Admixtures for Concrete
ASTM C 494	Standard Specification for Chemical Admixtures for Concrete
ASTM C 618	Standard Specification for Coal Fly Ash and Calcined Natural Pozzolan for Use in Concrete
ASTM C 685	Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing
ASTM C 1602	Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
ASTM C 1603	Standard Test Method for Measurement of Solids in Water
ASTM D 2061	Standard Test method of Strength of Zippers
ASTM D 4354	Practice for Sampling of Geotextiles for Testing
ASTM D 4491	Standard Test Methods for Water Permeability of Geotextiles by Permittivity
ASTM D 4533	Standard Test Method for Trapezoidal Tearing Strength of Geotextiles
ASTM D 4595	Test Method for Tensile Properties of Geotextiles by the Wide Width Strip Method
ASTM D 4632	Test Method for Breaking Load and Elongation of Geotextiles (Grab Method)
ASTM D 4751	Test Method for Determining Apparent Opening Size for a Geotextile
ASTM D 4759	Practice for Determining the Specification Conformance of Geotextiles
ASTM D 4873	Standard Guide for Identification, Storage, and Handling of Geotextiles
ASTM D 4884	Test Method for Seam Strength of Sewn Geotextiles
ASTM D 5199	Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes
ASTM D 5261	Test Method for Measuring Mass per Unit Area of Geotextiles

ASTM D 6241	Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 2-inch [50-mm] Probe
ASTM D 6449	Standard Method for Flow of Fine Aggregate Concrete for Fabric Formed Concrete

1.4 Terminology

For the purpose of these specifications, the following definitions shall apply:

1.4.1 Compaction:

The densification of a soil by means of mechanical manipulation.

1.4.2 Subgrade:

The ground surface usually specially prepared against which lining shall be placed. In cases where lining is to be retained the same shall be considered as subgrade.

1.4.3 Hydrotex™ Fabric Form:

The fabric forms are constructed of woven, double-layer synthetic fabric. HYDROTEX linings are installed by positioning fabric forms over the areas to be protected and then pumping, high-strength, fine aggregate concrete into the forms. The fabric forms can be placed and filled either underwater or in-the-dry. The high-strength, fine aggregate concrete is used in place of conventional concrete because of its pumpability, high-strength, impermeability, and absorption resistance.

1.4.4 Hydrotex™ Articulating Block (AB) Lining:

Hydrotex Articulating Block Linings consist of a series of compartments (blocks) linked by an interwoven perimeter and revetment cables. Ducts interconnect the compartments and high strength revetment cables are installed between and through the compartments and ducts. Once filled, the Articulating Block Linings become a mattress of pillow shaped, rectangular concrete blocks. The interwoven perimeters between the blocks serve as a hinge to permit articulation. The cables remain embedded in the concrete blocks to link the blocks together and facilitate articulation. Some relief of hydrostatic pressure is accomplished through the filtration bands formed by the interwoven perimeters of the blocks.

1.4.5 Baffle:

Baffles are flow-directing vertical geotextile walls constructed between fabric form sections layers. Baffles are an integral part of the fabric form design. Baffles are designed to support the panel section, determine the concrete area of the section and direct the flow of fine aggregate concrete for maximum efficiency.

1.4.6 Slide Fastener (Zipper):

A zipper or zipper like devise having two grooved plastic edges joined by a sliding tab or pull.

1.5 Submittals

1.5.1 The Contractor shall furnish the fine aggregate concrete manufacturer's certificates of compliance, mix design, fine aggregate gradation and fineness modulus for the fine aggregate concrete.

1.5.2 The Contractor shall furnish the fabric form manufacturer's certificates of compliance for the fabric forms. The Contractor shall also furnish the manufacturer's specifications, literature, shop drawings for the layout of the concrete lining panels, and any recommendations, if applicable, that are specifically related to the project.

1.5.3 Alternative fabric formed concrete lining materials may be considered. Such materials must be pre-approved in writing by the Engineer prior to the bid date. Alternative material packages must be submitted to the Engineer a minimum of fourteen (14) days prior to the bid date. Submittal packages must include, as a minimum, the following:

Material testing reports prepared by a certified geotextile laboratory attesting to the alternative fabric form material's compliance with this Specification. Material laboratory testing shall have been performed within ninety (90) days of the bid date.

PART 2:.0 PRODUCT

2.1 General - Fabric Formed Concrete Lining

Fabric formed concrete lining shall be Articulating Block (AB400LL) type with concrete blocks having finished nominal block dimensions of 22 inches x 14 inches, a finished average thickness of 4.0 inch, and a nominal mass per unit area of 45 lb/ft^2. Concrete blocks shall be interconnected with embedded longitudinal revetment cables in such a manner as to provide longitudinal and lateral binding of the finished articulating block mattress. The shear resistance of the concrete lining shall be a minimum of 26 lb/ft^2, as demonstrated by full scale flume testing.

2.2 Fabric Forms

The fabric forms for casting the concrete lining(s) shall be as specified, HYDROTEX® Articulating Block (AB400LL) fabric forms as manufactured by:

Synthetex, LLC; 5550 Triangle Parkway, Suite 220 Peachtree Corners, Georgia 30092
Tel: 800.253.0561 or 770.399.5051
E-Mail: info@synthetex.com

The fabric forms shall be composed of synthetic yarns formed into a woven fabric. Yarns used in the manufacture of the fabric shall be composed of polyester. Forms shall be woven with a minimum of 50% textured yarns (by weight). Partially-oriented (POY), draw-textured, and/or staple yarns shall not be used in the manufacture of the fabric. Each layer of fabric shall conform to the physical, mechanical and hydraulic requirements Mean Average Roll Values listed in Table 1.0. The fabric forms shall be free of defects or flaws which significantly affect their physical, mechanical, or hydraulic properties.

Table 1.0 PROPERTY REQUIREMENTS – HYDROTEX FABRIC [1, 2]

	Test Method	Units	MARV
Physical Properties			
Composition of Yarns	-	-	Polyester
Mass Per Unit Area (double-layer)	ASTM D 5261	oz/yd^2	13
Thickness (single-layer)	ASTM D 5199	mils	15
Mill Width (Woven)		inch	84
Mechanical Properties			
Wide-Width Strip Tensile Strength - MD | TD	ASTM D 4595	lbs/inch	300 | 350
Elongation at Break - MD | TD - Max.	ASTM D 4595	%	15 | 15

Trapezoidal Tear Strength - MD | TD	ASTM D 4533	lbs	150 **|** 175
CBR Puncture Strength	ASTM D 6241	lbs	1250
Mullen Burst Strength	ASTM D 3786 (Mod.)	psi	500
	Test Method	**Units**	**MARV Range**
Hydraulic Properties			
Apparent Opening Size (AOS)	ASTM D 4751	U.S. Standard Sieve	30 - 40
Flow Rate	ASTM D 4491	gal/min/ft^2	30 - 55

Notes:

1. Conformance of fabric to specification property requirements shall be based on ASTM D 4759.

2. All numerical values represent minimum average roll values (i.e., average of test results from any sample roll in a lot shall meet or exceed the minimum values). Lots shall be sampled according to ASTM D 4354.

2.2.1 Fabric forms shall be double-layer woven fabric joined together by narrow perimeters of interwoven fabric into a matrix of rectangular compartments. Cords shall connect the two layers of fabric at the center of each compartment. The cords shall be interwoven in two sets of four cords each, one set shall cross from the top layer to the bottom layer and the other from the bottom layer to the top layer. Each cord shall have a minimum breaking strength of 160 lbf when tested in accordance with ASTM D 2256. Fabric form compartments shall be offset in the lateral direction, to form a bonded concrete block pattern.

2.2.2 Fabric form compartments shall each have six ducts, two on each of the long sides and one on each of the short sides to allow passage of the fine aggregate concrete between adjacent compartments. The fine aggregate concrete filled, cross-sectional area of each duct shall be no more than 10 percent of the maximum filled cross-sectional area of the block lateral to the duct.

2.2.3 Revetment cables shall be installed in the longitudinal directions between the two layers of fabric. Two longitudinal cables, on approximately 12-inch centers, shall pass through each compartment in a manner which provides for the longitudinal and binding of the finished articulating block mattress. The cables shall enter and exit the compartments through opposing ducts.

2.2.4 Revetment cables shall be installed in the lateral direction between the two layers of fabric. One lateral cable shall pass through each compartment in a manner which provides for the lateral binding of the finished articulating block mattress. The lateral cables shall enter and exit the compartments through opposing ducts.

2.2.5 Revetment cables shall be Polyester Revetment Cables. Cables shall be constructed of high tenacity, low elongation, and continuous filament polyester fibers. Cable shall consist of a core constructed of parallel fibers contained within an outer jacket or cover. The weight of the parallel core shall be between 65% to 70% of the total weight of the cable. Longitudinal cables shall be nominally 0.25 inches in diameter and their rated breaking strength shall be not less than 3,700 lbs. and transverse cables shall be 0.25 inches in diameter and their rated breaking strength shall be not less than 3,700 lbs., or as specified by the Engineer.

Paragraph 2.2.5 is a standard guideline for the selection of revetment cables. The Engineer should consult with the Synthetex's engineering department for site specific revetment cable selections. Alternate cable strengths and constructions are available.

2.2.6 Mill widths of fabric shall be a minimum of 84 inches. Each selvage edge of the top and bottom layers of fabric shall be reinforced for a width of not less than 1.35 inches by adding a minimum of 6 warp yarns to each selvage construction. Mill width rolls shall be cut to the length required, and the double-layer fabric separately joined, bottom layer to bottom layer and top layer to top layer, by means of sewing thread, to form multiple mill width panels with sewn seams on not less than 80-inch centers.

2.2.7 Fabric form panels shall be factory-sewn, by jointing together the layers of fabric, top layer to top layer and bottom layer to bottom layer, into predetermined custom sized panels. Sewn seams shall be downward facing as shown on the Contract Drawings. All sewn seams and zipper attachments shall be made using a double line of U.S. Federal Standard Type 401 stitch. All seams sewn shall be not less than 100 lbf/inch when tested in accordance with ASTM D 4884. Both lines of stitches shall be sewn simultaneously and be parallel to each other, spaced between 0.25 inches to 0.75 inches apart. Each row of stitching shall consist of 4 to 7 stitches per inch. Thread used for seaming shall be polyester.

2.2.8 Baffles shall be installed at predetermined mill width intervals to regulate the distance of lateral flow of fine aggregate concrete. The baffles shall be designed to maintain a full concrete lining thickness along the full length of the baffle. The baffle material shall be nonwoven filter fabric. The grab tensile strength of the filter fabric shall be not less than 180 lbf/inch when tested in accordance with ASTM D 4632.

2.2.9 The fabric forms shall be kept dry and wrapped such that they are protected from the elements during shipping and storage. If stored outdoors, they shall be elevated and protected with a waterproof cover that is opaque to ultraviolet light. The fabric forms shall be labeled as per ASTM D 4873.

2.2.10 The Contractor shall submit a manufacturer's certificate that the supplied fabric forms meet the criteria of these Specifications, as measured in full accordance with the test methods and standards referenced herein. The certificates shall include the following information about each fabric form delivered:

Manufacturer's name and current address;
Full product name;
Style and product code number;
Form number(s);
Composition of yarns; and
Manufacturer's certification statement.

2.3 Fine Aggregate Concrete

Fine aggregate concrete consists of a mixture of Portland cement, fine aggregate (sand) and water, so proportioned and mixed as to provide a pumpable fine aggregate concrete.

The water/cement ratio of the fine aggregate concrete shall be determined by the ready-mix manufacturer, but generally should be on the order of 0.65 to 0.70. The pumping of fine aggregate concrete into the fabric forms causes a reduction in the water content by filtering excess mixing water through the permeable fabric. The reduction of mixing water substantially improves the water/cement ratio of the in-place fine aggregate concrete thereby increasing its strength and durability. The sand/cement ratio should be determined by the ready-mix manufacturer and should be on the order of 2.4:1.

The consistency of the fine aggregate concrete delivered to the concrete pump should be proportioned and mixed as to have a flow time of 9-15 seconds when passed through the ¾-inch [19 mm] orifice of the standard flow cone that is described in ASTM C6449-99. Additional Pozzolan and/or admixtures may be used with the approval of the Engineer-in-charge. The water/cement ratio varies with the exact granulometry of the fine aggregate (sand) and should be determined by the ready-mix manufacturer using the above referenced flow cone.

The Contractor should demonstrate the suitability by placing the proposed fine aggregate concrete mix into three (3) 2-inch concrete cubes. The mix should exhibit a minimum compressive strength of 3500 psi at 28 days, when made and tested in accordance ASTM C109/C109M-13.

With a typical loss of approximately 15% of the total mixing water, 27 ft^3 of pumpable fine aggregate concrete will reduce to approximately 25 ft^3 of hardened concrete. The mixing water reduction will also result in an increase of approximately 8% in the sand and cement per cubic foot of concrete. The range of fine aggregate concrete mix proportions provided in Table 2.0 has been developed under a variety of field conditions.

Table 2.0 Typical Range of Mix Proportions

Material	Mix Proportions lb/yd^3	After Placement Mix Proportions lb/yd^3
Cement	750-850	805-915
Sand	2120-2030	2290-2190
Water	540-555	460-470
Air	As Required	As Required

2.3.1 Components

2.3.1.1 Portland Cement

Portland cement should conform to ASTM C 150/150M, Type I, II or V. Pozzolan grade fly ash may be substituted for up to 35% of the cement as an aid to pumpability. (The pumpability of fine aggregate concrete mixes containing course sand is improved by the addition of fly ash.) Pozzolan, if used, should conform to ASTM C 618, Class C, F or N.

2.3.1.2 Fine Aggregate (sand)

Fine aggregate should consist of suitable clean, hard, strong and durable natural or manufactured sand. It should not contain dust, lumps, soft or flaky materials, mica or other deleterious materials in such quantities as to reduce the strength and durability of the concrete, or to attack any embedded steel, neoprene, rubber, plastic, etc. Motorized sand washing machines should be used to remove impurities from the fine aggregate. Fine aggregate having positive alkali-silica reaction should not be used. All fine aggregates should conform to ASTM C33/C33M-13. The fine aggregate should not have more than 45% passing any sieve and retained on the next consecutive sieve of those shown in Table 3.0. The fineness modulus of fine aggregate should neither be less than 2.3 nor greater than 3.1. Fine aggregate with grading near the minimum for passing the No. 50 and No. 100 sometimes have difficulties with workability or pumping. The additions of entrained air, additional cement, or the addition of an approved mineral admixture to supply the deficient fines, are methods used to alleviate such difficulties.

ASTM C33/C33M-13 defines the requirements for grading and quality of fine aggregate for use in fine aggregate concrete and is for use by a contractor as part of the purchase document describing the material to be furnished.

Table 3.0 Grading Requirement for Fine Aggregate

Sieve	Percent by Weight Passing the Sieve

9.5-mm (3/8-in.)	100
4.75-mm (No. 4)	95 to 100
2.36-mm (No. 8)	80 to 100
1.18-mm (No. 16)	50 to 85
600-μm (No. 30)	25 to 60
300-μm (No. 50)	5 to 30
150-μm (No. 100)	0 to 10
75-μm (No. 200)	0 to 3

Fine aggregate failing to meet these grading requirements can be utilized provided that the supplier can demonstrate to the specifier that fine aggregate concrete of the class specified, made with fine aggregate under consideration, will have relevant properties at least equal to those of fine aggregate concrete made with same ingredients, with the exception that the referenced fine aggregate will be selected from a source having an acceptable performance record in similar fine aggregate construction.

2.3.1.3 Water

Water used for mixing and curing should be clean and free from injurious amounts of oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to concrete.

Potable water is permitted to be used as mixing water in fine aggregate concrete without testing for conformance with the requirements of ASTM C1602/C1602M-12.

ASTM C1602/C1602M-12 covers the compositional and performance requirements for water used as mixing water in hydraulic cement fine aggregate concrete. It defines sources of water and provides requirements and testing frequencies for qualified individual or combined water sources.

2.3.2 Plasticizing and Air Entraining Admixtures

Grout fluidifier, water reducing or set time controlling agents may be used as recommended by their manufacturers to improve the pumpability and set time of the fine aggregate concrete.

Any air entraining agent or any other admixture may be used, as approved, by the Engineer-in-charge to increase workability, to make concrete impervious and more durable. Air entraining admixture should conform to ASTM C494/C494M and ASTM C260/C260M, respectively. Mixes designed with 5% to 8% air content will improve the pumpability of the fine aggregate concrete, freeze-thaw and sulfate resistance of the hardened concrete.

2.4 Ready-Mixed Concrete

The basis of standard specifications for ready-mixed concrete should be ASTM C94/C94M-13a.

2.4.1 Ordering

The contractor should require the manufacturer to assume full responsibility for the selection of the proportions for the concrete mixture, the contractor should also specify the following:

1. Requirements for compressive strength as determined on samples taken from the transportation unit at the point of discharge. Unless otherwise specified the age at test should be 28 days.

2. That the manufacturer, prior to the actual delivery of the fine aggregate concrete, furnish a statement to

the contractor, giving the dry mass of cement and saturated surface-dry-mass of fine aggregate and quantities, type, and name of admixtures (if any) and the water per cubic yard or cubic metre of fine aggregate concrete that will be used in the manufacture. The manufacturer should also furnish evidence satisfactory to the contractor that the materials to be used and proportions selected will produce fine aggregate concrete of the quality specified.

2.4.2 Mixing and Delivery

Ready-mixed fine aggregate concrete should be mixed and delivered to the point of discharge by means of one of the following combinations of operation:

Central-Mixed Concrete is mixed completely in a stationary mixer and transported to the point of delivery in a truck agitator, or a truck mixer operating at agitating speed, or in non-agitating equipment meeting the requirements of Section 13 of ASTM C94/C94M-13a. The acceptable mixing time for mixers having capacity of 1 yd^3 or less is one (1) minuet. For mixers of greater capacity, this minimum should be increased 15 seconds for each cubic yard [cubic metre] of fraction thereof of additional capacity.

Shrink-Mixed Concrete—Concrete that is first partially mixed in a stationary mixer, and then completely in a truck mixer, should conform to the following: The time for the partial mixing should be the minimum required to intermingle the ingredients. After transfer to a truck mixer the amount of mixing at the designated mixing speed will be that necessary to meet the requirements for uniformity of concrete.

Truck-Mixed Concrete—Concrete that is completely mixed in a truck mixer, 70 to 100 revolutions at the mixing speed designated by the manufacturer to produce the uniformity of concrete.

No water from the truck water system should or elsewhere should be added after the initial introduction of mixing water for the batch except when on arrival to the project site the flow rate of the fine aggregate concrete is less than 9 seconds. If the flow rate is less than 9 seconds obtain the desired flow rate within 9 to 15 seconds with a one-time addition of water. A one-time addition of water is not prohibited from being several distinct additions of water provided that no fine aggregate concrete has been discharged except for flow testing. All water additions should be completed within 15 minutes from the start of the first water addition. Such addition should be injected into the mixer under such pressure and direction of flow to allow for proper distribution within the mixer. The drum should be turned an additional 30 revolutions, or more if necessary, at mixing speed to ensure that a homogenous mixture is attained. Water should not be added to the batch at any later time.

Discharge of fine aggregate concrete should be completed within 1 1/2 hours after the introduction of mixing water to the cement and fine aggregate. This limitation may be waived by the contractor if concrete is of such flow after 1 1/2 hours' time has been reached that it can be placed, without the addition of water to the batch. In hot weather, or under conditions contributing to rapid stiffening of the fine aggregate concrete, a time less than 1 1/2 hours is permitted to be specified by the contractor. *Depending on the project requirements the technology is available to the manufacture to alter fresh fine aggregate properties (such as setting time or flow.) On some projects the manufacturer may request changes to certain fresh fine aggregate concrete properties due to the distance or projected transportation time between the batch plant and the point of delivery.*

Fine aggregate concrete delivered in cold weather should have the minimum temperature indicated in Table 4.0. The maximum temperature of fine aggregate concrete produced with heated aggregate, heated water, or both, should at no time during its production or transportation exceed 90 °F.

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Table 4.0 Minimum Fine Aggregate Temperature as Placed

Section Size, inch	Temperature, min, °F
< 12	55
12—36	50

2.4.3 Sampling for Uniformity

The fine aggregate concrete should be discharged at the normal operating rate for the mixer being tested, with care being exercised not to obstruct or retard the discharge by an incompletely opened gate or seal. As the mixer is being emptied, individual samples should be taken after discharge of approximately 15% and 85% of the load. *No samples should be taken before 10% or after 90% of the batch has been discharged. Due to the difficulties of determining the actual quantity of fine aggregate discharged, the intent is to provide samples that are representative of widely separated portions, but not the beginning and end of the load.*

2.4.4 Batch Ticket Information

The manufacturer of the concrete should furnish to the contractor with each batch of fine aggregate concrete before unloading at the site, a delivery ticket with the following information:

- Name of ready-mix company and batch plant, or batch plant number.
- Serial number of ticket,
- Date,
- Truck number,
- Specific designation of job (name and location),
- Specific call or designation of the concrete in conformance with that employed in project specifications,
- Amount of fine aggregate concrete in cubic yards,
- Time loaded or of first mixing of cement and fine aggregate, and
- Amount of water added to the fine aggregate concrete by the contractor, at site, or the contractor's designated representative and their initials.

The following information, for certification purposes, required by the project specifications should be furnished:

- Type, brand, and amount of cement,
- Class, brand, and amount of coal fly ash, or raw or calcined natural pozzolans,
- Type, brand, and amount of admixtures.
- Source and amount of each metered or weighted water,
- Information necessary to calculate the total mixing water. Total mixing water includes water on fine aggregates, batch water (metered or weighted) including ice batched at the plant, wash water retained in the mixing drum, and water added by the truck operator from the mixer tank,
- Amount of fine aggregate,
- Ingredients certified as being previously approved, and
- Signature or initials of manufacturer's representative.

2.3 **Geotextile Filter Fabrics**

2.4.1 The geotextile filter fabrics shall be composed of synthetic fibers or yarns formed into a nonwoven or woven fabric. Fibers and yarns used in the manufacture of filter fabrics shall be composed of at least 85% by weight of polypropylene, polyester or polyethylene. They shall be formed into a network such that the filaments or yarns retain dimensional stability relative to each other, including selvages. The geotextile shall be free of defects or flaws which significantly affect its mechanical or hydraulic properties.

2.4.2 The geotextile filter fabric must be permitted to function properly by allowing relief of hydrostatic pressure; therefore fine soil particles shall not be allowed to clog the geotextile. The geotextile filter fabric shall be as specified elsewhere in the Contract Specifications. Final acceptance of the geotextile filter fabric by the Engineer shall be based on project specific soil information, provided by the Contractor/Owner. The geotextile filter shall meet the minimum physical requirements listed in Table 5.

2.4.3 The geotextile filter fabric shall be kept dry and wrapped such that they are protected from the elements during shipping and storage. If stored outdoors, they shall be elevated and protected with a waterproof cover that is opaque to ultraviolet light. The fabric forms shall be labeled as per ASTM D 4873.

Table 5.0 PROPERTY REQUIREMENTS – FILTER FABRIC

	Test Method	Units	Minimum Value
Mechanical Properties			
Grab Tensile Strength	ASTM D 4632	lbf	180 (in any principal direction)
Elongation at Break	ASTM D 4632	%	50 max. (in any principal direction)
Trapezoidal Tear Strength	ASTM D 4533	lbf	75 (in any principal direction)
Puncture Strength	ASTM D 4833	lbs	105 (in any principal direction)
CBR Puncture Strength	ASTM D 6241	lbs	475 (in any principal direction)
Hydraulic Properties			
Apparent Opening Size (AOS)	ASTM D 4751	US Sieve	As Specified Elsewhere in the Contract Specifications
Permittivity	ASTM D 4491	sec^{-1}	As Specified Elsewhere in the Contract Specifications
Flow Rate	ASTM D 4491	$gal/min/ft^2$	As Specified Elsewhere in the Contract Specifications

Notes:

1. Conformance of fabric to specification property requirements shall be based on ASTM D 4759.
2. All numerical values represent minimum average roll values (i.e., average of test results from any sample roll in a lot shall meet or exceed the minimum values). Lots shall be sampled according to ASTM D 4354.

PART 3.0: DESIGN REQUIREMENTS

3.1 Certification (Open Channel Flow)

3.1.1 Fabric formed concrete lining will only be accepted when accompanied by documented full-scale hydraulic flume performance characteristics that are derived from tests under controlled flow conditions. Test guidelines shall conform to testing protocol as documented in “Hydraulic Stability of Fabric Formed Concrete Lining and Mat Systems During Overtopping Flow.”

3.1.2 The average thickness, mass per unit area and hydraulic resistance of each concrete lining shall withstand the hydraulic loadings for the design discharges along the structure(s). The stability analysis for each concrete lining shall be accomplished using a factor-of-safety methodology. A minimum factor of safety of 1.3 shall be required or higher as determined by lock conditions or critical structures.

3.2 Performance (Open Channel Flow)

3.2.1 The Contractor shall provide to the Engineer calculations and design details, provided by the manufacturer or a professional engineer, attesting to the suitability of each fabric formed concrete lining for the purpose contemplated. Each concrete lining shall be accepted only when accompanied by the documented hydraulic performance characteristics derived from full-scale flume tests performed under controlled flow conditions.

PART 4.0: CONSTRUCTION AND INSTALLATION REQUIREMENTS

4.1 Site Preparation - Grading

4.1.1 Areas on which fabric forms are to be placed shall be constructed to the lines, grades, contours, and dimensions shown on the Contract Drawings. The areas shall be graded and uniformly compacted to a smooth plane surface with an allowable tolerance of plus or minus 0.2 feet from bottom grade, as long as ponding does not occur, and plus or minus 0.2 foot from a side slope grade as long as humps or pockets are removed.

4.1.2 The areas shall be free of organic material and obstructions such as roots and projecting stones and grade stakes shall be removed. Where required by the Contract Specifications, soft and otherwise unsuitable subgrade soils shall be identified, excavated and replaced with select materials in accordance with the Contract Specifications. Where areas are below the allowable grades, they shall be brought to grade by placing compacted layers of select material. The thickness of layers and the amount of compaction shall be as specified by the Engineer.

4.1.3 Excavation and preparation of aprons as well as anchor, terminal or toe trenches shall be done in accordance with the lines, grades, contours, and dimensions shown on the Contract Drawings.

4.1.4 The terminal edges of the fabric form lining should be keyed into the subgrade to the lines, grades, and dimensions shown on the Contract Drawings.

4.2 Inspection

Immediately prior to placing the fabric forms, the prepared area shall be inspected by the Engineer, and no forms shall be placed thereon until the area has been approved.

4.3 Geotextile Filter Fabric Placement

4.3.1 The geotextile filter baric shall be placed directly on the prepared area, in intimate contact with the subgrade, and free of folds or wrinkles. The geotextile filter fabric shall be placed so that the upstream roll of fabric overlaps the downstream roll. The longitudinal and transverse joints will be overlapped at least two (2) feet. The geotextile will extend at least one (1) foot beyond the top and bottom concrete lining termination points, or as required by the Engineer.

4.3.2 A geotextile filter fabric, as specified elsewhere, shall be placed on the graded surface approved by the Engineer.

4.4 Fabric Form Placement

4.4.1 Factory assembled fabric form panels shall be placed over the geotextile filter fabric and within the limits shown on the Contract Drawings. Perimeter termination of the fabric forms shall be accomplished through the use of anchor, flank and toe trenches, as shown on the Contract Drawings. When placing panels an allowance for approximately 10% contraction of the form in each direction which will occur as a result of fine aggregate concrete filling. The contractor shall gather and fold the additional slope direction fabric form in the anchor trench to be secured in such a manners as to be gradually released as fabric forms contract during filling. The contractor shall gather the additional transverse direction fabric form at each baffle for self-release during filling.

4.4.2 Adjacent fabric form panels shall be joined in the field by means of sewing or zippering closures. Adjacent panels shall be joined top layers to top layer and bottom layer to bottom. All field seams shall be made using two lines of U.S. Federal Standard Type 101 stitches. All sewn seams shall be downward facing.

4.4.3 When conventional joining of fabric forms is impractical or where called for on the Contract Drawings, adjacent forms may be overlapped a minimum of 3 ft to form a lap joint, pending approval by the Engineer. Based on the predominant flow direction, the upstream form shall overlap the downstream form. In no case shall simple butt joints between forms be permitted. Simple butt joints between panels shall not be allowed.

4.4.4 Expansion joints shall be provided as shown on the Contract Drawings, or as specified by the Engineer.

4.4.5 Immediately prior to filling with fine aggregate concrete, the assembled fabric forms shall be inspected by the Engineer, and no fine aggregate concrete shall be pumped therein until the fabric seams have been approved. At no time shall the unfilled fabric forms be exposed to ultraviolet light (including direct sunlight) for a period exceeding five (5) days.

4.5 **Fine Aggregate Concrete Placement**

4.5.1 Following the placement of the fabric forms over the geotextile filter fabric, fine aggregate concrete shall be pumped between the top and bottom layers of the fabric form through small slits to be cut in the top layer of the fabric form or manufacturer supplied valves. The slits shall be of the minimum length to allow proper insertion of a filling pipe inserted at the end of a 2-inch I.D. concrete pump hose. Fine aggregate concrete shall be pumped between the top and bottom layers of fabric, filling the forms to the recommended thickness and configuration.

Holes in the fabric forms left by the removal of the filling pipe shall be temporarily closed by inserting a piece of fabric. The fabric shall be removed when the concrete is no longer fluid and the concrete surface at the hole shall be cleaned and smoothed by hand.

4.5.2 Fine aggregate concrete coverage for AB400LL shall net 75 ft^2/yd^3 (see Section 2.3).

4.5.3 Fine aggregate concrete shall be pumped in such a manner that excessive pressure on the fabric forms is avoided. Consultation with the fabric form manufacturer with regard to the selection of grout/concrete pumps is recommended.

4.5.4 Cold joints shall be avoided. A cold joint is defined as one in which the pumping of the fine aggregate concrete into a given section of form is discontinued or interrupted for an interval of forty-five (45) or more minutes.

4.5.5 The sequence of fine aggregate concrete shall be such as to ensure complete filling of the fabric formed concrete lining to the thickness specified by the Engineer. The flow of the fine aggregate concrete shall first be directed into the lower edge of the fabric form and working back up the slope, followed by redirecting the flow into the anchor trench.

4.5.6 Prior to removing the filling pipe from the current concrete lining section and proceeding to the fine aggregate concrete filling of the adjacent lining section, the thickness of the current lining section shall be measured by inserting a length of stiff wire through the lining at several locations from the crest to the toe of the slope. The average of all thickness measurements shall be not less than the specified average thickness of the concrete lining. Should the measurements not meet the specified average thickness, pumping shall continue until the specified average thickness has been attained.

4.5.7 Excessive fine aggregate concrete that has inadvertently spilled on the concrete lining surface shall be removed. The use of a high-pressure water hose to remove spilled fine aggregate concrete from the surface of the freshly pumped concrete lining shall not be permitted.

4.5.8 Foot traffic will not be permitted on the freshly pumped concrete lining when such traffic will cause permanent indentations in the lining surface. Walk boards shall be used where necessary.

4.5.9 After the fine aggregate concrete has set, all anchor, flank and toe trenches shall be backfilled and compacted flush with the top of the concrete lining. The integrity of the trench backfill must be maintained so as to ensure a surface that is flush with the top surface of the concrete lining for its entire service life. Toe trenches shall be backfilled as shown on the Contract Drawings. Backfilling and compaction of trenches shall be completed in a timely fashion to protect the completed concrete lining. No more than five hundred (500) linear feet of pumped concrete lining with non-completed anchor, anchor, flank, or toe trenches will be permitted at any time.

PART 5.0: Method of Measurement

The fabric formed concrete erosion control lining shall be measured by the number of square feet or yards computed from the lines and cross sections shown on the Contract Drawings or from payment lines established in writing by the Engineer. This includes fabric forms, fine aggregate concrete, and filter fabric used in the aprons, overlaps, anchor, terminal, or toe trenches. Slope preparation, excavation and backfilling, and bedding are separate pay items.

Hydrotex products are manufactured and sold by:

SYNTHETEX

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ATTACHMENT 6 – ACB PRODUCT LITERATURE FROM SHORETEC, LLC (2016)

CP: **SS** Date: **5/19/2017** APC: **GDN** Date: **5/19/2017** CC: **MWS** Date: **5/19/2017**

Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

SHORETEC, LLC
510 O' Neal Lane
Baton Rouge, LA 70819

225-408-1444 (phone)
225-408-1445 (fax)
shoretec.com (web)

Distributed By:

SHORETEC®

SHOREBLOCK®

SHOREBLOCK® SD blocks of different heights and weights can be assembled to provide a castellated cover layer for a higher coefficient of hydraulic friction or improved wave energy absorption and retention.

SHOREBLOCK® SD is a flexible, interlocking matrix of concrete blocks of uniform size, shape and weight connected by a series of cables which pass longitudinally through preformed ducts in each block. SHOREBLOCK® SD revetment systems combine the favorable aspects of lightweight blankets and meshes, such as porosity, flexibility, vegetation encouragement and habitat enhancement with non-erodible, self-weight and high tractive force resistance of a rigid lining.

SHOREBLOCK® SD has proven to be an aesthetic and functional alternative to rip-rap, poured in place concrete and other heavy-duty, erosion protection systems. SHOREBLOCK® SD is easy to install, therefore, can dramatically reduce overall project costs. More specifically, when compared to other systems, life-cycle costs have been reduced because SHOREBLOCK® SD is a permanent system and saves on subsequent maintenance expenses.

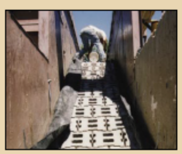
Research and Design

SHOREBLOCK® SD is the most durable, effective and environmentally-friendly erosion control revetment method of fighting severe erosion problems. SHOREBLOCK® SD mats are available in eight foot widths in lengths up to 40 feet. Mats can be joined to achieve greater lengths. Different sizes of SHOREBLOCK® SD are available depending on the severity of the application. In most markets, Articulated Concrete Blocks (ACBs) are competitive in cost to 12" diameter (or greater) rock (or rip-rap) placed in an 18" or greater blanket thickness, are competitive with gabion mattresses and ACBs are typically more economical than poured in place concrete.

ACBs were successfully tested by the U.S. Bureau of Reclamation and U.S. Federal Highway Administration (FHWA-RD-89-199). The Corps of Engineers has used ACBs on numerous designs for both channel and shoreline stability. Comprehensive wave tank testing was evaluated in 1983 at Oregon State University. ACB installations have been performing successfully since 1980.

SHOREBLOCK® SD DESIGN ADVANTAGES

- Each block has an open area of up to 20% to allow for superior hydrostatic pressure relief and ecologically pleasing vegetative cover.
- Interlocking cabling allow greater flexibility through the axes of articulation — conforms better to ground contours and settlement.
- Prefabricated mats offer quick installation, even underwater.
- Tests have shown that the force needed to remove a block from a revegetated cover layer may be equal to 20 times the weight of the block.



SHOREBLOCK® SD has been successfully tested by Colorado State University, in accordance with the hydraulic performance testing protocol established by the U.S. Federal Highway Administration. (FHWA-RD-89-199).

MIN. DENSITY (IN AIR) (Lbs./Ft.^{3})		MIN. COMPRESSIVE STRENGTH (PSI)		MAX. WATER ABSORPTION (Lbs./Ft.^{3})	
AVE. OF 3 UNITS	INDIVIDUAL UNIT	AVE. OF 3 UNITS	INDIVIDUAL UNIT	AVE. OF 3 UNITS	INDIVIDUAL UNIT
130	125	4,000	3,500	9.1	11.7

* Unit weight and density values may vary due to availability of local materials.

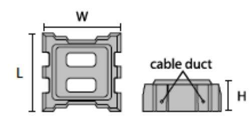
Specifications

Fabrication of a SHOREBLOCK® SD mat is accomplished by threading corrosive resistant steel or special synthetic cable in one direction through a series of blocks. Cables are then secured to the mattress with corrosive resistant hardware. Cables are sized to provide a 5 to 1 cable strength to mat weight ratio to ensure safe handling while providing extraordinary strength in the system. Longitudinal cables are looped together at the ends of each row of blocks in the mat assembly for easy handling and anchoring. The open cells of SHOREBLOCK® SD comprise about 20% of the mat area.

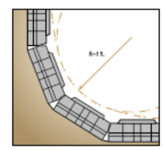
OPEN CELL							
BLOCK CLASS	DIMENSIONS IN.			BLOCK		UNIT COVERAGE Sq. Ft.	OPEN AREA
	H	W	L	Unit Weight Lbs.	System Weight Lbs./Sq. Ft.		
SD-400 OC	4.00	15.50	17.40	50-57	28-32	1.78	20%
SD-475 OC	4.75	15.50	17.40	62-71	35-40	1.78	20%
SD-600 OC	6.00	15.50	17.40	81-94	46-53	1.78	20%
SD-800 OC	8.00	15.50	17.40	108-118	61-67	1.78	20%
SD-900 OC	9.00	15.50	17.40	120-138	68-78	1.78	20%

CLOSED CELL							
BLOCK CLASS	DIMENSIONS IN.			BLOCK		UNIT COVERAGE Sq. Ft.	OPEN AREA
	H	W	L	Unit Weight Lbs.	System Weight Lbs./Sq. Ft.		
SD-400 CC	4.00	15.50	17.40	66-73	37-41	1.78	10%
SD-475 CC	4.75	15.50	17.40	78-89	43-50	1.78	10%
SD-600 CC	6.00	15.50	17.40	94-108	53-61	1.78	10%
SD-800 OC	8.00	15.50	17.40	125-135	71-76	1.78	10%
SD-900 CC	9.00	15.50	17.40	145-167	82-98	1.78	10%

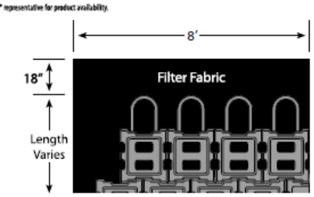
*The SD Series denotes Single Directional Cable System. Note: Additional block styles may be available in some areas. Check with your local SHORETEC® representative for product availability.



SHOREBLOCK® SD units are manufactured in accordance with ASTM C90, C140 and D6684-04.



Each block is interconnected by flexible cables, providing articulation between adjacent blocks.



Woven monofiliments are preferred over nonwoven geotextiles. The soil's particle size (among other factors) will ultimately determine the fabric selection.

Features & Benefits

DURABILITY
SHOREBLOCK® SD will not suffer loss of function due to chemical degradation, UV degradation, biological degradation, vandalism or aging throughout its design life.

STABILITY
SHOREBLOCK® SD has the necessary strength characteristics to resist displacement due to imposed tractive forces and wave loads and the necessary strength to resist both lateral displacement and vertical uplift.

ACCEPTABILITY
SHOREBLOCK® SD becomes part of the landscape and the local ecosystem. Its construction is free of hazardous projections thus offering opportunities for recreation as native grasses are quick to germinate in the soil-filled cells.

AFFORDABILITY
The SHOREBLOCK® SD System is engineered to ensure comprehensive project design, and high quality components at 20-50% lower than alternative erosion control methods.

CP: **SS** Date: **5/19/2017** APC: **GDN** Date: **5/19/2017** CC: **MWS** Date: **5/19/2017**

Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

Specifications for Shoreblock SD Closed Cell (CC) Articulated Concrete Blocks (ACBs), SHORETEC LLC

	5 PSF	6 PSF	7 PSF	8 PSF	9 PSF	10 PSF
SD 475 CC	12.0	11.7	11.4	11.1	10.9	10.6
SD 600 CC	13.7	13.4	13.4	12.8	12.5	12.2
SD 900 CC	15.9	15.6	15.6	14.9	14.6	14.1

* MAX VELOCITY AT 1.5 FOS ON 0.005% BED SLOPE

	5 PSF	6 PSF	7 PSF	8 PSF	9 PSF	10 PSF
SD 475 CC	13.8	13.5	13.2	13.0	12.8	12.6
SD 600 CC	15.7	15.3	15.1	14.9	14.6	14.3
SD 900 CC	18.3	17.9	17.6	17.3	17.0	16.8

* MAX VELOCITY AT 1.2 FOS ON 0.005% BED SLOPE

Specifications for Articulated Concrete Blocks provided as per communication between with Shoretec, LLC (2016)

ATTACHMENT 7 – HAND CALCULATION OF ICE THICKNESS

CP: **SS** Date: **5/19/2017** APC: **GDN** Date: **5/19/2017** CC: **MWS** Date: **5/19/2017**

Client: **RD Group** Project: **Gowanus Canal Superfund Site** Project No: **HPH106A**

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Written by: Mark Schillinger Date: 16 / 12 / 20 Reviewed by: Shaunya Sood Date: 16 / 12 / 22

Client: RD Group Project: Gowanus Canal Project/Proposal No.: HPH106A Task No.: 40.04

Ice Scour Potential

$X = \alpha U_j^{1/2}$

where:

X = ice thickness (inches)

α = 0.20 TO 0.40 for "small sheltered river"

U_j = air freezing index of 440°F-days for a 100-yr recurrence interval at Central Park, NYC

$X = 0.20 \times (440°\text{F-days})^{1/2} = \underline{4.2}$ inches

$X = 0.40 \times (440°\text{F-days})^{1/2} = \underline{8.4}$ inches

These values are significantly lower than the depth of the Canal. Thus, scour from ice is not considered to be a significant design concern